1911

Old School Catalog 1911, Engineering

Valparaiso University

THE

ENGINEERING ANNUAL

Published Annually

by the

Civil Engineering Society

of

Valparaiso University

Number 1

May, 1911

Price Fifty Cents

Valparaiso, Indiana

THE

ENGINEERING ANNUAL

Published Annually

by the

Civil Engineering Society

of

Valparaiso University

Number 1

May, 1911

Price Fifty Cents

Valparaiso, Indiana

UNIVERSITY PRESS
VALPARAISO, INDIANA

1911 BOARD OF EDITORS

EDITORIAL STAFF

Arthur D. Buzby, 1911 . . . Editor-in-Chief

S. B. Ehrenrich, 1911 . . Assistant Editor-in-Chief

BUSINESS STAFF

Emil A. Hurme, 1912 . . . Business Manager

Hyman M. Shulman, 1912 . Assistant Business Manager

EXCHANGE AND DISTRIBUTION STAFF

A. F. Dougal, 1912 . . . . Exchange Editor

. . Assistant Exchange Editor

ADVISORY BOARD

Prof. M. E. Bogarte . Dean of School of Civil Engineering

R. C. Yeoman . . Professor of Civil Engineering

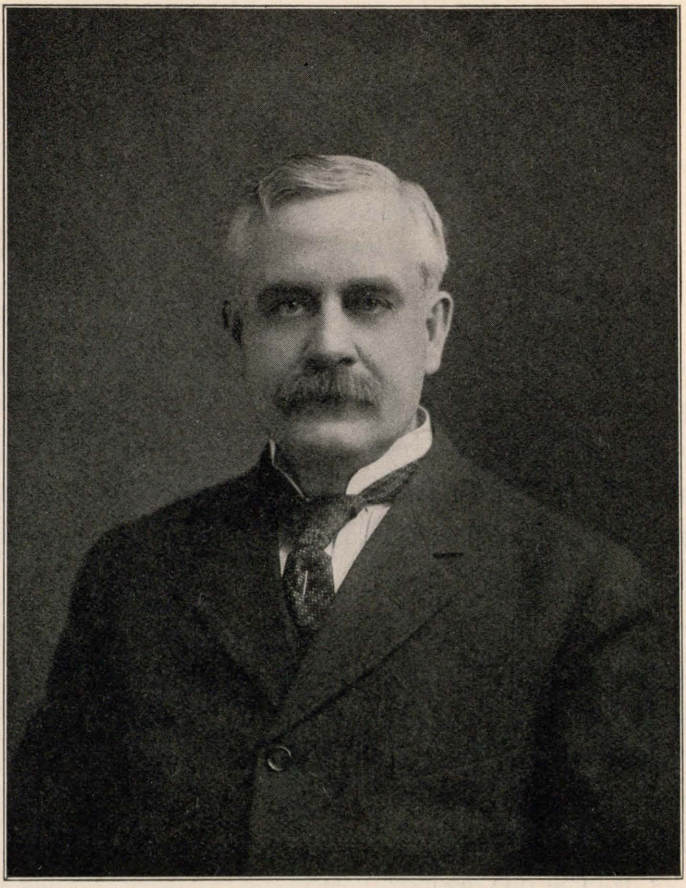
TABLE OF CONTENTS

PREFACE

The purpose of the Engineering Annual may be stated as follows: First:--To bring before the student body engineering information of a lasting value, which information may or may not have been previously published. Second:--To stimulate original thought and investigation among the engineering students by providing them with a fitting medium to communicate their ideas to those most interested in them.

The board of editors present this, the first issue of the Annual, with mingled feelings of apprehension and satisfaction. Apprehension lest it may not receive the cordial reception they have hoped for it, and satisfaction in that its publication witnesses the completion of a task thought to be too great for a Society in its infancy and a Board of Editors without experience. The articles, with all due modesty, may be said to be of considerable merit, and it is expected that several of them will be of value to Junior and Senior year Engineering Students.

The Board of Editors wish to thank the contributors and advertisers, without whose hearty support this publication would have been impossible.



MARTIN EUGENE BOGARTE

Dean of Civil Engineering, Valparaiso University

Born May 3, 1855. Educated in Northwestern Ohio Normal School, Boston University and Massachusetts Institute of Technology. He has been Professor in charge of Mathematics since the foundation of Valparaiso University, and was for many years in charge of the Department of Oratory. He is now the head of the Civil Engineering Department, and the growth of this department and the high standard of efficiency maintained in it are due to his untiring efforts.

THE FOURTH DIMENSION

M. E. Bogarte

Many thinkers from very remote times have entertained the suspicion, at least, that the possibilities of space are not exhausted by the dimensions length, breadth, and thickness, as appears to us.

Kant imagined that space might contain more than three dimensions, and the investigations of the modern non-Euclidean geometers have done much to clear the way for the conception of space of four, or even more than four, dimensions.

Let us endeavor to define "dimension" and see how through our definition we may by generalization be led to such a conception. In geometry, we think of a point as position without extension. If we consider the shortest distance between two points we have a straight line. This presents the simplest case of the one-dimension magnitude—the length. We may conceive the straight line to be generated by a point moving in an invariable direction.

If now at the extremities of a straight line we draw two other straight lines, each perpendicular to it and on the surface of the paper, and then imagine the first line to move parallel to its original position so that its ends shall follow the two perpendiculars, it will generate a plane. This plane will be a square, and will represent a magnitude of two dimensions, viz., length and breath. The square presents four points and four lines.

Now we may suppose that at each of its four points perpendiculars are drawn to the sides meeting at those points, and of equal length to the sides. These four lines will also be perpendicular to the surface of the square. Also suppose the square to move parallel to its original position so that each of its angular points shall follow these perpendiculars. By such a motion it will generate a solid—a space of three dimensions, viz., length, breadth, and thickness. This cube presents 8 points, 12 lines, and 6 planes.

Let us note that the line receives from the point 2 points—its extremities; that the square receives from the line 4 points and four lines, the latter resulting from the line in its original position, the 2 lines generated by its extremities, and the line in its final position; that the cube derives from the moving square 8 points—the 4 in the original, and four in the final position, 12 lines—the 4 in the original position of the generating square, the 4 in its final position, and the 4 generated by its four points in the course of its transition; 6 planes,—1 from the first, and 1 from the last position of the generating square, and 4 traced by its 4 sides as it moved.

Now we have reached the limit of our experience, but must we stop here? As the line was moved in a direction perpendicular to its length to form the surface, the surface in a direction perpendicular to each of the two lines meeting at one of the points, to form the cube, can the last be moved in a direction perpendicular to each of the three lines meeting at one of its points, to generate a new figure whose relation to the cube shall be analogous to the relation of the cube to the square, or of the square to its generating line?

We can conceive of no such motion, but we are three-dimensional creatures, live and think in an apparently three-dimensional world, and there are "more things in heaven and earth, Horatio, than are dreampt of in your philosophy."

So, while it is true that we cannot conceive of the four-dimensional space which would result from a motion of the cube similar to that imposed on the line and the square, we can by a little attention discover its corresponding properties.

Since the cube has 8 points in its original position and 8 points in its final position, the fourth-dimensional figure should have 16 points. The cube has 12 lines in its initial position, 12 in its final, and each of its 8 points has generated a line, hence the corresponding fourth-dimensional figure has 32 lines. Also, at the beginning, the cube has 6 planes, at the end 6, and in its motion each of its 12 lines has traced a plane, making 24 planes in the fourth-dimensional figure. Lastly, this figure will be bounded by 8 cubes: for, to the 2 cubes presented by the initial and final position of the moving cube, we must add the 6 generated by the motion of its 6 faces. Hence our hypothetical figure of four dimensions is bounded by 16 points, 32 lines, 24 planes, and 8 cubes.

As intimated above, it is not sufficient to deny the existence of this hypothetical fourth-dimensional figure because we can form no mental image of it.

We, being three-dimensional, invariably think of it as some sort of a solid, or made up of solids.

But it is not a solid. A section of it is a solid, as the section of a solid is a plane, and we can conceive of one of its sections, just as an imaginary two-dimensional being could see but a plane section of a solid.

It may help us if we consider the perplexity of such a creature if asked to conceive of a solid.

Suppose the home of a two-dimensional intelligence to be on the surface of still water. If a solid, an inverted cone, for example, be thrust into the water this intelligence would see it, first as a point, then as a gradually lengthening line. At the instant of complete immersion it would vanish from his sight and his world. Assure him that it yet exists, and that to see it he has but to look obliquely to his own plane, and you are speaking in terms as incomprehensible to him as a direction perpendicular to each of the three intersecting edges of a cube is incomprehensible to you. To him all space is included in the direction of the plane in which he has his being. Place such a creature in a polygon, and he is as completely imprisoned as you would be in a six-walled dungeon. For him to step over the boundary of his polygonal prison and escape would be as impossible as for you to step through the walls of your cell to freedom. As a three-dimensional being could look into the inside of a two-dimensional being, so a fourth-dimensional being could look inside a three-dimensional.

Thus we are led to some sufficiently startling conclusions: for example, a being of four dimensions could extract a treasure locket from a steel safe without disrupting its walls; could take a kernel from a nut and leave the shell intact; could turn a hollow rubber ball inside out without tearing the ball; could appear and disappear at will,—a thought of which many spiritualistic humbugs have made great capital.

CONCRETE FORMS

SOME NOTES ON THE PRACTICAL DESIGN OF FORMS FOR CONCRETE WITH SPECIAL REFERENCE TO THE DEVELOPMENT AND USE OF THE TIMBER TEE-COLUMN.

BY R. C. YEOMAN, C. E.*

A superintendent of Reinforced Concrete Construction remarked to the writer that a small, simple and practical handbook on forms for concrete would meet a great demand among concrete foremen and carpenters. It is not the purpose of this paper to cover that field, but only to suggest some of the ideas worthy of such a book Just how much theory to present and how much to leave to experience is the hardest problem for writers on this subject. It is possible to design forms with considerable mathematical accuracy, which will insure maximum economy of material, time and safety to workmen. To present simple methods for correct theoretical design, in such a way that the average man can use them without understanding the theory of their derivation, is the principal purpose to be accomplished.

Lack of space will permit only a small part of ordinary wooden form design to be considered; therefore most patented and special forms now on the market will be omitted.

The writer acknowledges that much of the material that follows is taken from the actual office practice of the National Bridge Company of Indianapolis, where he was once employed. And in co-operation with D. B. Luten, President of the Company, and Professor H. C. Berry of the University of Pennsylvania, the original ideas of the scheme were worked out. Since then the writer has performed a number of laboratory experiments, devised new formulas and added several new parts until its usefulness is more than doubled. However the experiments are not complete and all data and derivations will be omitted until a later publication.

An ideal form for concrete shall be (1) strong, (2) smooth surface, (3) water tight, (4) non-absorbent, (5) easily worked or constructed, (6) light weight, (7) cheap, (8) and easily removed without injuring itself, the concrete, or workmen.

Wood in its natural state had fulfilled most of the above requirements. Its weak points are usually overcome by some simple treatment, for instance: It is very absorbent, but a little oil or paint applied to it practically eliminates this difficulty. Numerous species of timber are used for concrete forms, their selection depending on the above eight qualities. Usually cheapness, strength, and ease of working are the most important factors. Short leaf yellow pine has been used

*Professor of Civil Engineering, Valparaiso University.

widely and will be the species selected for basis of future calucula-tions. Hemlock is often used here (Valparaiso, Ind.) and might be substituted for short leaf yellow pine in the same design.

In presenting tables or diagrams for design it is assumed that the material is uniform in quality. It is impossible and impracticable to incorporate in the diagrams the solution of the dozen or more defects a piece of timber may have, so the designer is called upon to solve special problems to overcome these defects, if there are any. The average man, who will use such a scheme of design cannot do this, and must therefore depend upon a uniform quality of material or risk failure.

A wooden form is made up of a number of beams and columns joined together with nails, bolts, clamps, or wires. Simple beams and

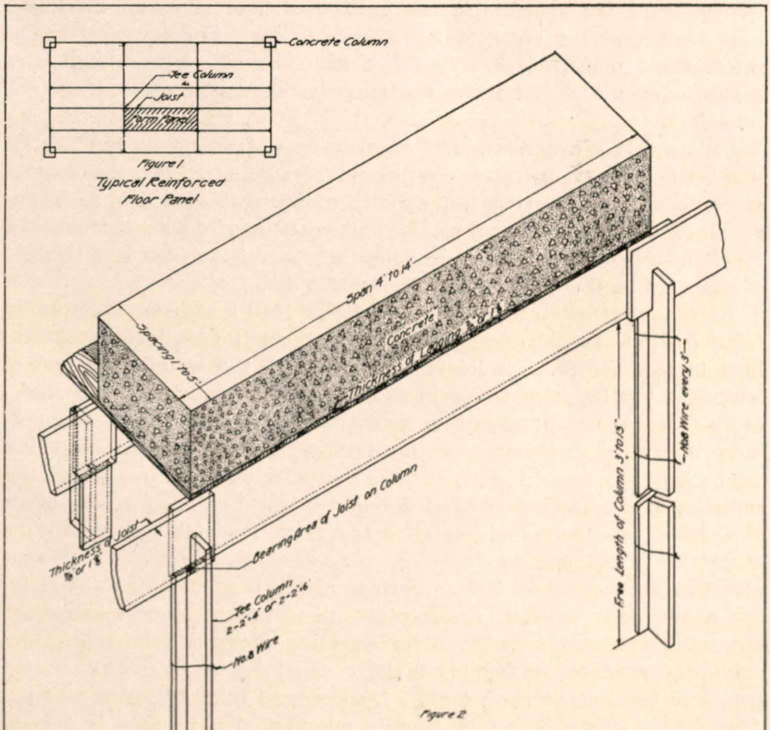


Figure 2
FORM PANEL
FOR
TYPICAL REINFORCED CONCRETE FLOOR
Designed by R.C. Yerman Traced by C.B.P.
April-1911

columns are easily designed, but when they are dependent on the strength of joints, made with nails, bolts, etc., the problem becomes complex.

The particular problem presented here is a design of a form to support reinforced concrete slab. The same scheme is also successfully used in reinforced concrete arch forms, where it had its origin. The floor slab is chosen for its simplicity of construction. A typical reinforced concrete floor panel is shown in figure 1. This will be a panel, say 12½ ft. by 24 ft. center to center columns. The forms are built in smaller panels according to methods to be explained. The cross-hatched portion is enlarged in figure 2. The arrangement is apparently very simple, a block of concrete on a floor supported by paneled joists and columns. The concrete is usually uniform in thickness and weighs from 130 to 140 lbs. per cubic foot, adding about three per cent in weight for steel, it is 135 to 145 lbs. For uniformity and safety use 150 lbs. cu. ft. To select the most economical lengths of spans for flooring and joists the diagram shown in Plate I will be consulted.

Plate I is a combination of diagrams so arranged to perform all the mathematical operations of design graphically. At the bottom of the plate are the lagging curves, whose ordinates (on AA') are equal to the span of the lagging or spacing of joists in feet and abscissae (on AB) are equal to depth of concrete in inches or weight of concrete in lbs. per sq. ft. The middle panel of the plate contains a more complex diagram. The abscissae (A'B') are the same as in lagging diagram (AB) but ordinates on the left (A'L') represent the load in lbs. per foot on joists or in figure 2 the weight per foot of length of the block of concrete shown on the form. The lines radiating from A' are arranged to give spacing of joists; by the aid of these it will be seen later that the total weight of the concrete block (Fig. 2) or any panel load is obtained on abscissae ML just above the radiating lines. At the right of the center is a diagram for the design of depth and thickness of joists for any given load (shown on ML) as of columns and one for area of cross-grain bearing. The column abscissal and length in feet as ordinates (MB').

Above the line ML are plotted two sets of curves; one for strength of columns and one for area of cross-grain bearing. The column curves are made from (ML) panel loads as ordinates and length in feet as abscissae (MP). Curves for 1 in. x 4 in., 2 in. x 4 in., 1 in. x 6 in., and 2 in. x 6 in. timbers are plotted to be used in design of sway and wye bracing. The Tee-columns, made up of 2—2 in. x 4 in. or 2—2 in. x 6 in. are a special design for panel support and will be treated in full later. The crushing cross grain curves are used to investigate safety of joint between column and joists in bearing. Its ordinates are equal column loads in pounds and abscissae are equal to safe bearing area for that load in square inches.

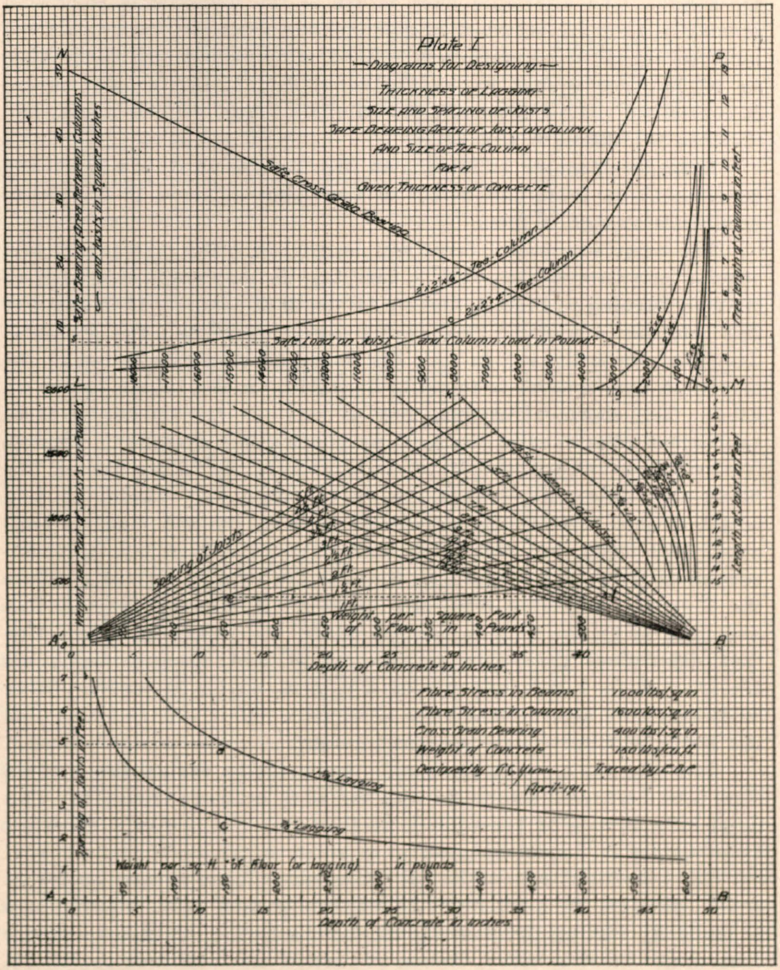


Plate I.
—Diagrams for Designing—
Thickness of Lagging
Size and Spacing of Joists
Safe Bearing Area of Joist on Column
And Size of Tee-Column
For a
Given Thickness of Concrete
Safe Cross-Grain Bearing
Tee-Column
Safe Load on Joist — and Column Load in Pounds
Safe Bearing Area Between Columns and Joists in Square Inches
Free Length of Columns in Feet
Weight per Foot of Joists in Pounds
Spacing of Joists
Length of Joists
Length of Joist in Feet
Weight per Square Foot of Floor in Pounds
Depth of Concrete in Inches
Fibre Stress in Beams 1000 lbs/sq.in
Fibre Stress in Columns 1600 lbs/sq.in
Cross Grain Bearing 400 lbs/sq.in
Weight of Concrete 150 lbs/cu.ft
Designed by R.C. Yinn
Traced by C.H.F.
April-1911
Lagging
Spacing of Joists in Feet
Weight per sq.ft. of Floor (or lagging) in pounds
Depth of Concrete in Inches

PLATE I.

The allowable fiber strees in beams is assumed at 1,000 lbs. per square inch. This is low but the water from the concrete weakens the timber sometimes one-half. Also live loads, such as occur in placing concrete, are often excessive and require part of the allowance for safety. The allowable fiber stress in the columns is somewhat greater as they are assumed to be more free from the water. Sixteen hundred pounds per square inch is used. Cross-grain bearing is taken as 400 lbs. per square in. as the usual safe strength. All lumber is considered planed although the columns might well be rough and increase the strength and safety.

Example—Assume building plan Fig. 1 to be 12 ft. 5 x 24 ft. and concrete 12 in. thick and height of floor above ground 10 ft. Select economical design of form.

The most economical form will be one, in which every part is a regular commercial size and is carrying its maximum safe load or stress. Turning to Plate I, find on AB depth of concrete 12 in., project this point on each of the two lagging curves to points c and d respectively, thence to the left to AA'; for the ⅞ in. thickness (point c) the maximum safe spacing is found to be 2.6 ft., for the 1⅝ in. (point d), 4.9 ft. The choice of spacing may or may not fall on one of these values but should come as near as possible for economy. Proceed from c and d upward to A'B' to point e where the 2.5 ft. spacing line intersects; follow the dashed line to the right to f where the 8 ft. length of joist crosses, thence upward to g on LM and read the total panel load 3,000 lbs. One joist must carry this load and its cross-section is next to be selected. Retrace the dashed line to h where the 8' length abscissae from MB' crosses. H falls on the safe side and near to 1⅝ in. x 12 in. section curve, which will be used. This then determines the joists in size, length, and spacing, which are 1⅝ in. x 12 in. x 8 ft. and spaced 2 ft. 6 in. center to center. Other selections may also be made and compared, but it will be found that the above will prove as economical as any.

The columns are next designed. Again follow dashed line upward from h to i, where it intersects with the 10 ft. ordinate from MP. This point falls outside of Tee-column made of two 2 in. x 4 in.'s, therefore two 2 in. x 6 in.'s must be used.

The bearing area between the joists and column is equal to 1⅝ in. x 6 in. plus 1⅝ in. x 1⅝ in. or 10.8 square inches. The required area is found by tracing the dashed line from g to the intersection of the cross-grain bearing curve, thence to the left to s and read on LN 7½ square inches, which is less than the actual area as is required for safety.

If however sway bracing and ties are introduced the length can be reduced one-half or to five feet free length, as shown in fig. 3. Then to design the column for five feet, retrace from i to the five foot ordinate at point j. Running to the left from j to intersect with the two

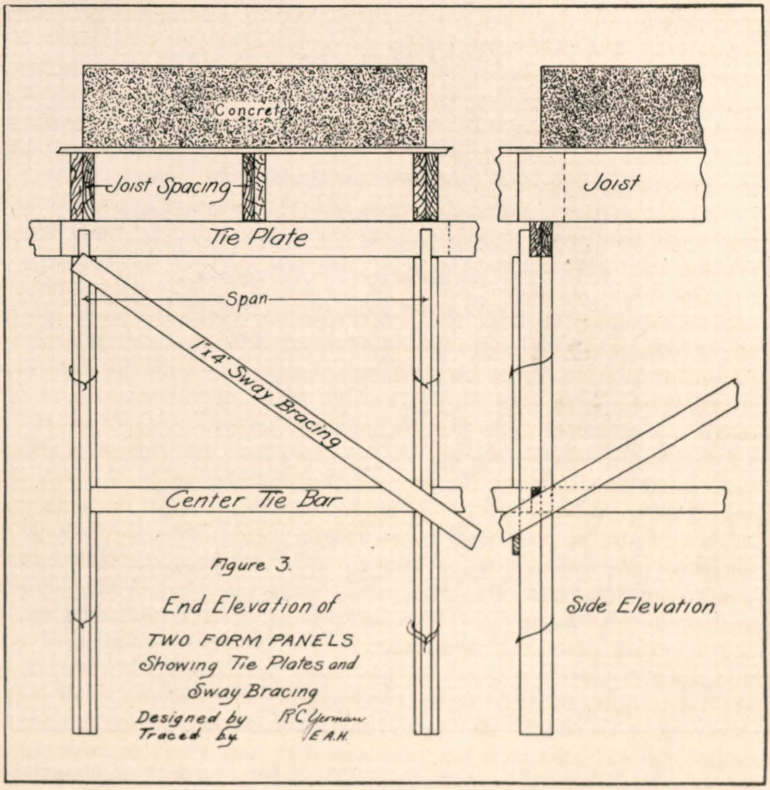
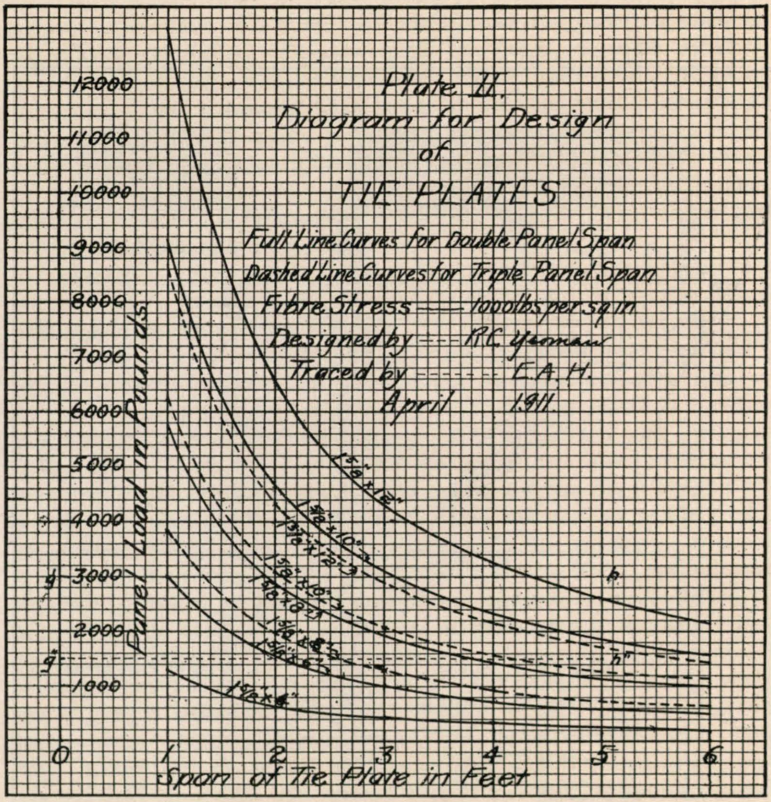


Figure 3.
End Elevation of
TWO FORM PANELS
Showing Tie Plates and
Sway Bracing

2 in. by 4 in. Tee-column curve at o, then to k on LM, it is found that a five foot column will carry 8,000 lbs. Two panels can be combined as in fig. 3, thus loading column to 6,000 lbs. The size of the tie plates are determined on Plate II. Beginning at 3,000 lbs. load or g′, proceed to the right to five ft. span h′. There is no curve above it; therefore no single timber listed is strong enough. To use two timbers start at (½ (3,000) or 1,500 lbs.) load or g″, proceed to right to 5 ft. span or h″. The full line curve just above it is for 1⅝ in. by 10 in. tie, which will be selected. (Note: Full line curve is used when tie supports two panels, dashed line curves when supporting three panels.)



The two designs will now be compared for economy of material.

The first will require per panel:

2—2 in. x 12 in. x 10 ft. ....20.0 board feet for Columns.
2—2 in. x 12 in. x 10 ft. ....20.0 board feet for Joists.
5—1 in. x 6 in. x 8 ft. ....20.0 board feet for Flooring
1—1 in. x 4 in. x 16 ft. .... 5.3 board feet for Bracing
Total ....65.3 board feet

The second:

2—2 in. x 4 in. x 10 ft. ....13.3 board feet for Column
1—2 in. x 12 in. x 10 ft. ....20.0 board feet for Joist
1—2 in. x 10 in. x 7 ft. ....12.0 board feet for Tie Plate
1—1 in. x 4 in. x 16 ft. .... 5.3 board feet for Sway Bracing
4—1 in. x 6 in. x 8 ft. ....20.0 board feet for Flooring
1—2 in. x 4 in. x 5 ft .... 3.3 board feet for Tie Bar
Total ....73 9 board feet

The first uses less material and less number of pieces, which requires less labor and is therefore cheaper and best. Possibly, if three panels could be combined in one column, the second method would prove more economical. This solution will be left to the reader.

Thus it is seen that the design is worked out without the knowledge of higher mathematics. This method has been used by boys just graduated from common or high schools. It takes a few days of teaching for them but they soon become quite proficient and reliable in the work.

With the aid of the diagram the work is so quickly done that, several combinations of spacing, spans, sizes, etc., can be tried until one is found in which every member is loaded to the maximum. However, this might require odd sizes and lengths which would increase the waste of material and labor in cutting to more than offset the saving in actual timber necessary to carry the loads. Therefore, the commercial sizes of timbers available must be considered and all dimensions should be as near simple integral quantities as possible to facilitate the workman in cutting and using the material economically.

No allowance has been made for nailing or fastening the joints. This would ordinarily be a great factor in the design. The estimates on strength of nailed joints are so unreliable and so difficult that the tendency is to eliminate them wherever possible. Also the timber is so badly injured in breaking these joints, when the forms are removed, that the waste becomes excessive. Clamps and wires are becoming more popular because they are easily removed and do not injure the timber. The design in fig. 2 makes use of wire and a few nails for binding together of parts only. The nails do not carry any load direct and therefore are not considered in the diagram.

THE TEE-COLUMN. (See Figure 2.)

The Tee-column was designed and patent applied for by D. B. Luten several years ago. He has used it successfully in Reinforced Concrete Arch forms since. Its design is unique in embodying so many of the requirements for an ideal column to be used in concrete forms. (1) It is simple and cheap in construction; (2) the tee-section increases the radius of gyration over the rectangular section adding stiffness; (3) the wiring of members makes them easily removed, and (4) when the wires are cut the separate parts buckle, lowering the structure slightly and allowing the concrete to settle and assume its load gradually, still giving necessary support until safety is assured against sudden collapse, if concrete be defective. This method will take the place of wedges commonly used in arch forms.

The two pieces forming the column will be named to aid in further discussion. The member forming the stem of the T is called the Bearing Member and the cross-bar the Stiffening Member. These names are taken from the special function they originally performed.

In the original design the joists rested on the Bearing Member only and were nailed to the other. It was thought by the designer that each member would carry one-half the load, and the early formulas and designs followed this assumption. By actual test the Bearing Member carried about 90% of the load, which was nearly double what it would carry alone. The explanation is as follows: The nails in the Stiffening Member carried 8 to 10% of the load direct to that member as determined by nail bearing test, so the rest of the excess load carried, must by elimination be accredited to its stiffening effect on the Bearing Member. This discovery made a great change in the design. It was the practice to use 2—2"x4" or 2—2"x6" columns according to the load, and when an intermediate load was to be carried a 2"x4" and 2"x6" were combined. The 2"x6" was made the Stiffening Member and as originally planned was depended on to carry its share of the load in proportion to its area of cross-section. The nails would not carry any more in this case than before; but it would be expected that it would give more stiffness than the 2"x4". By actual experiment it was found that a 1"x4" as a stiffener for a 2"x4" would develop all the stiffness the wires would transmit, and the 2"x6" was mostly wasted. Therefore the best combinations for this old method of loading are 2—2"x4"'s, 2"x4" and 2"x6", 2—2"x6" and 2"x6" and 2"x8". Where there are two sizes the larger should be the Bearing Member. Although 1"x4"'s can be used once with economy they are so light that handling breaks them up badly, and should not be used where forms are to be repeated.

The results of a few tests* made by a squad of students in Testing Materials at Valparaiso University showed that another design (see figure 2) was still better. It was thought that if in the first case one member was stiffened by the other to carry double load, then when each was directly loaded and stiffened by the other, they ought to carry nearly four times the load borne by one of them. This was found to be approximately true for the specimens tested. The number of tests being so small just the conclusions are given. In another year a more complete series of tests will be run and results published. The formulas used for plotting the diagrams are figured with a greater safety factor than necessary until a more extended investigation can be run.

* Tests of Timber Tee-columns by Messrs. H. F. Black, E. A Hurme, and A. D. Buzby.

AN EXAMPLE OF SLOW-BURNING CONSTRUCTION

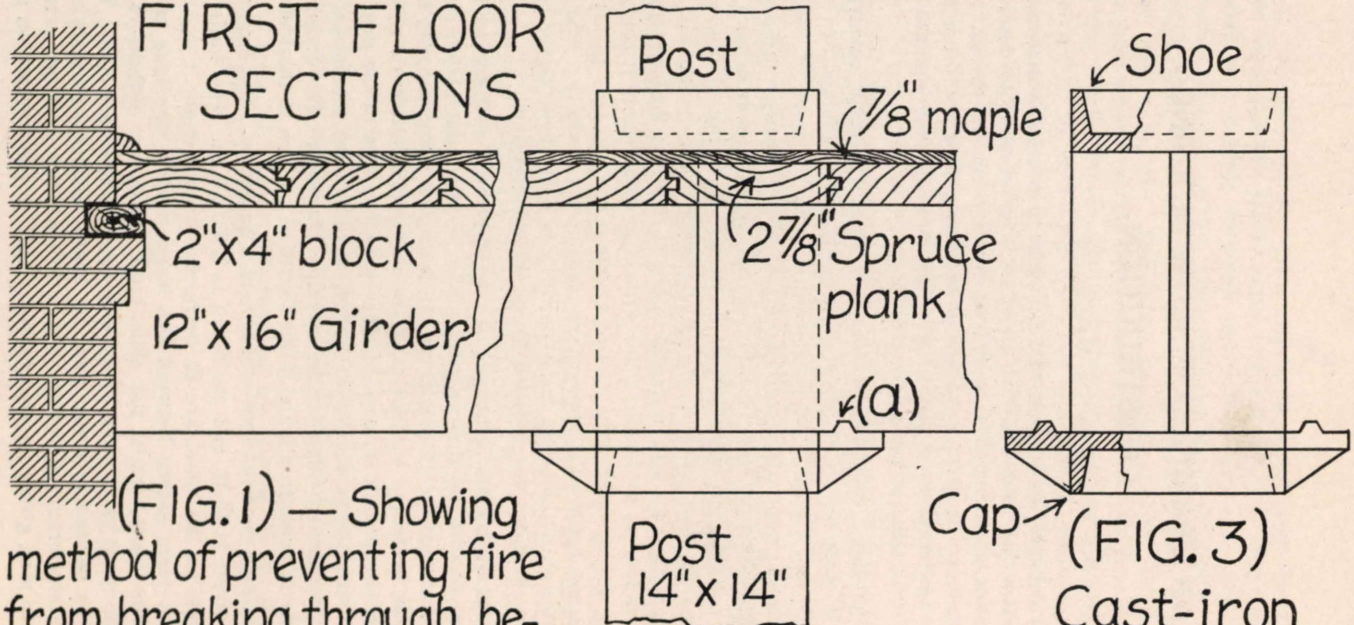
By N. M. Stineman

IN THIS paper no attempt will be made to present a discussion of an "engineering problem," but an effort will be made to describe with sufficient clearness a type of construction which, though frequently employed in certain classes of buildings, does not often fall under the observation of people who are not directly engaged in designing or building structures of this kind.

"Slow-burning" construction does not mean fire-proof construction; but it must be remembered that "fire-proof" buildings are not always fire-proof, as we have repeatedly seen in the work of conflagrations which swept up everything before them, whether fire-proof or not. Slow-burning construction of the type presented here is especially suitable for ware-houses, light-manufacturing plants, etc., in cities which have efficient fire departments. This point will be brought out further on, where it will be shown how the design is made in such a manner as to confine a fire within the floor where it started, at least for a time sufficiently long to enable a fire company to arrive upon the scene before the flames have made much headway. The insurance rates on buildings of this type will run only about 25 per cent higher than the rates on so-called fire-proof construction.

The original working drawings of this building were made by the writer, who was at the time in the employ of the construction company which erected it. In preparing the floor plan accompanying this paper it was necessary to omit many of the smaller dimension lines and details, but enough remains to give one a fair idea of the general plan. The lettering is also much larger, in proportion to the size of the plan, than the lettering on the working drawings. The drawings were originally made to a scale of one-fourth of an inch to the foot.

The building contains four stories and a basement. The frontage along the street is 60 ft. and the depth is 132 ft. 3 in. The walls, as will be seen on the plan, are of brick-pier construction; i. e., the heavy brick piers shown between the large window openings carry all the floor loads, while the thin brick walls, or curtain-walls, between the piers carry very little aside from their own weight. The floor load is carried to these piers by heavy wooden beams, which in turn are supported on wooden posts. The window openings extend from pier to pier, giving an abundance of light to the workers on any



(FIG. 1) — Showing method of preventing fire from breaking through between floor and wall

(FIG. 2)

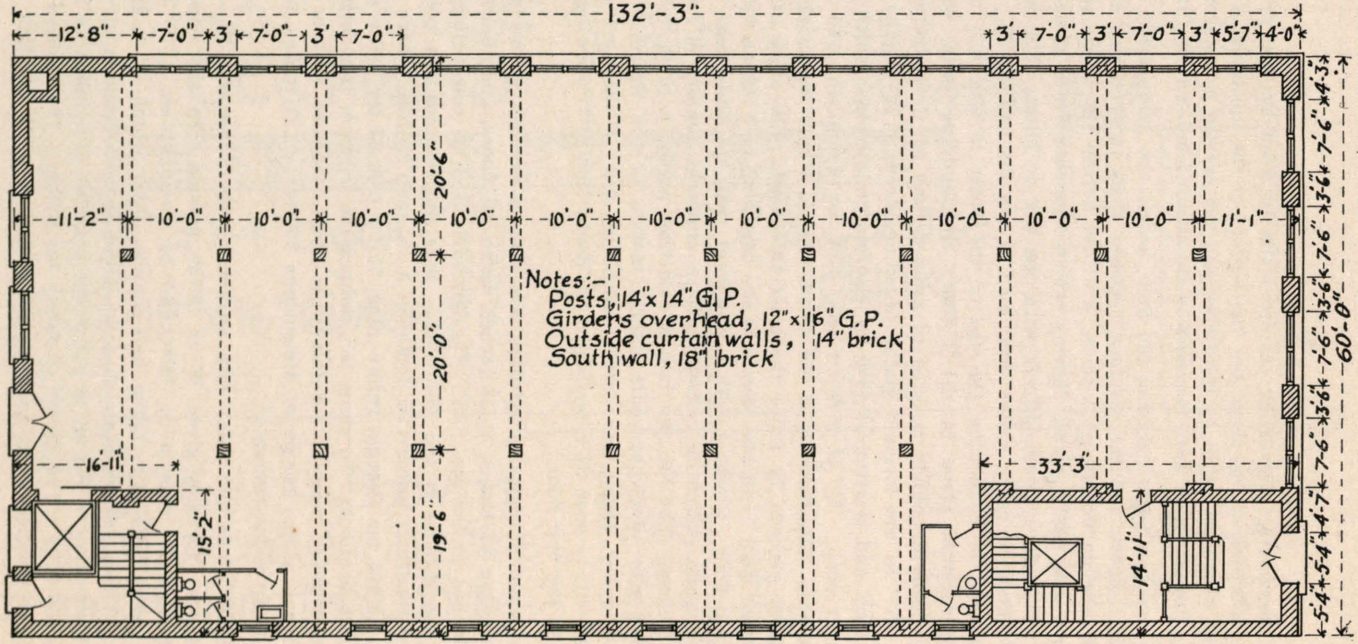
(FIG. 3) Cast-iron pintle

part of the floors. The brick curtain walls between the top of the window openings of one floor and the bottom of the openings of the next floor above are 14 in. in thickness, and are carried by reinforced concrete lintels which were moulded on the job and hoisted into place.

The wooden posts carrying the floor beams are arranged in two rows 20 ft. apart, and the distance between posts in each row is 10 ft. The wooden beams, or girders, are laid across the 20-ft. span, and carry the heavy plank flooring which stretches from girder to girder. This does away entirely with the use of joists. The posts were designed to carry a live floor load of 150 lbs. per sq. ft. The formula used in computing the size of the posts is a simple compression formula given in the Toronto city building laws, where the safe load in pounds = area of cross-section × C ÷ factor of safety. "C" in the formula is the crushing strength in pounds per square inch, and is given as 5000 for Georgia yellow pine, which was the timber used. "C" is 4000 for oak, 3500 for white pine and 3000 for hemlock. The formula is not to be applied where the height of the column exceeds 12 times the least thickness. The sizes of the posts range from 14 inches square in the basement to 8 inches square on the top floor. Starting with a basement post, the base rests in a cast-iron shoe, which is imbedded in a concrete footing 4 feet 6 inches square. The top fits into a cast-iron pintle, a plate with a projecting rim which prevents lateral motion of the timber. The pintle, however, forms one casting with the shoe of the next post above, as will be seen by referring to the detail (figure 3) showing the casting in the first floor.

The wooden beams, or girders, are also of Georgia yellow pine, the size being 12 inches × 16 inches under all floors, and 10 inches × 12 inches under the roof. The formula used in computing the size of the girders is the following: Safe uniform load in pounds = 2 × breadth × square of depth × A ÷ span in feet, where "A" is given as 100 for Georgia yellow pine, 75 for oak, 65 for white pine, and 55 for hemlock. To show the comparison between this formula and the bending moment formula $p = Me/I$, the two will be applied to one of the beams of dimensions 20 feet × 12 inches × 16 inches, and results compared.

Taking W as the total safe uniform load in pounds, the city building-laws formula gives $W = 30720$ pounds. In the bending moment formula, take $p = 1200$, and by computation, $I = 4096$, $e = 8$ inches, and $M = 51200$ foot-pounds, from which $W = 20480$ pounds. These results show that the building-laws formula gives a rather high value to W; or, to put it another way, p is taken as 1800. This leaves quite a low factor of safety for timber. The ends of the girders are notched so as to fit over the flanges of the casting (fig.



Scale 1/4 in. = 1'-0"

GROUND FLOOR PLAN

MINERVA MFG. CO. BUILDING

2 and fig. 3), thus preventing lateral motion, while a notch in the bottom of the timber which fits over a lug in the casting, shown at (a), fig. 2, prevents longitudinal motion. The end of the girders which rests on the wall is placed upon a 12 inch × 18 inch × ½ inch cast-iron plate which is imbedded in the brick.

The basement floor is made of a 6-inch layer of cinders covered with 4 inches of concrete and one inch of cement-and-sand mortar. All the floors above the basement are built of 2⅞ inch spruce plank, tongue-and-grooved, and covered with one thickness of ⅞ inch maple flooring. By computation it may be shown that for the spruce plank "p" in the above formula is about 935. The plank, of course, takes the place of joists in ordinary construction. The roof is nearly flat, being sloped merely enough to drain off the water, and consists of 1⅞ inch spruce plank covered with a 4-ply coat of tar and gravel.

It will thus be seen that no light, thin, or inflammable material is used in the construction of the floors or any of the supports, hence a fire breaking out anywhere within the building would require considerable time to break through the heavy flooring, or to weaken the heavy posts and girders to the point of collapsing. In figure 1 is shown the plan which was employed to prevent a fire from breaking through between the plank flooring and the brick wall. The scheme merely consists of corbeling several courses of brick below the floors.

The boiler-room is located in a corner of the basement and is surrounded by a brick wall, with a reinforced concrete floor overhead. This is done as a precaution against the possibility of loss from a fire in the boiler-room. Hemmed in by a brick wall and concrete floor and ceiling a fire starting in this room would not get very far if discovered within a reasonable time. The floor of the boiler-room is made two feet lower than the remainder of the basement, for the purpose of giving more head room. A brick fire-proof vault was constructed in another part of the basement. The wall is divided into two parts, consisting of an outer part of fire-clay brick, 14 inches in thickness, an inner part 4 inches thick, and a 2-inch air space between. The ceiling of the vault is made of a heavy reinforced concrete slab.

The front staircase is equipped with a passenger elevator in addition to the stairway, and a large freight elevator is placed in the rear staircase. Both staircases are separated from the rest of the building by brick walls, and fire-doors are placed at all openings leading to the main part of the building. The large fire-door leading from the freight elevator is suspended from cast-iron wheels running on an inclined track, and is balanced by a weight attached to a cord containing a fusible link. This link is composed of a material which fuses under a comparatively low temperature, and should the door be open at the time of a fire, the heat would soon burn out the link, thereby breaking the cord and allowing the door to close by the force of gravity.

The brick used in the building is a fairly good quality of red face-brick, laid in cement mortar with one-fourth inch joints. High class pressed brick is not easily obtainable in Toronto, as the local clays are not suitable for the best work, or else the process of manufacture is not well understood. The long shipment and the import duty on pressed brick sent from the United States renders the price prohibitive for ordinary work. The foundation up to 6 inches above grade is a plain concrete wall 27 inches thick, and the footings are 3 feet 6 inches wide. The front entrance is made of artificial stone supplied by a local company, and is a very good imitation of cut limestone. The cornice is made of galvanized iron, painted the color of the stone in the front entrance. The design of the cornice is rather elaborate—entirely too elaborate to harmonize with the remainder of the building—but the owner was determined to have an attractive cornice, and as he was unwilling to spend very much money for it, galvanized iron was about the only material which could be used. This material is the cheapest for the purpose, but also the least satisfactory.

The building was erected in 1907 for the Minerva Manufacturing Co., Toronto, Canada, manufacturers of various articles of ladies' wearing apparel. The total cost, including the builder's commission of 10 per cent., was $49,000; hence it may be well to do a little further computing and determine the probable cost of such a building per unit of area or unit of volume. There are two methods in common use for making a rough estimate of the cost of a building. One method consists in estimating it at a certain amount per square foot of floor space, including the basement. The other is the method of placing it at so much per cubic foot of volume. In determining the volume the usual practice is to regard the height as the distance from the base of the foundation to the top of the parapet wall, if the roof is flat, or to a point midway between the cornice and the ridge, if the roof is sloping. Upon this basis we may summarize the cost of this building as follows:

Square feet of floor space	39,675
Cubic feet of volume	39,675
Total cost	$49,000.00
Cost per square foot of floor space	1.24
Cost per cubic foot of volume	0.10

Of course there are so many local conditions governing the cost of a building that the foregoing statement should be regarded merely as a method and not as a definite key to be applied in all cases.

Everything else being equal, a building costs more in a large city than the same building would cost in a small town. Cost of labor, cost of material, ease of transportation, character of excavations, distance to shipping facilities and many other factors, must all be taken into consideration from a local viewpoint.

Summing up the proposition as a whole, and considering it from the standpoint of first cost, efficiency, durability, safety and insurance, it is the honest belief of the writer that the building here described is well worthy of the student's consideration if he is ever put into the position where it is left to him to choose or recommend the form of construction to be adopted.

THE ORIGIN OF THE EARTH.

BY LEE F. BENNETT

Professor of Geology and Mineralogy, Valparaiso University.

We are all interested in the problem of the earth's origin. It is fundamentally one of the most important problems presented for study, because of its bearing directly and indirectly upon our ideas concerning the origin and destiny of the human race.

Philosophers and scientists have seen the necessity of a rational solution of the problem, and it is interesting to note that their attempts have resulted in much the same kind of a solution. Kant, the German philosopher, after a study of the relationship of the known planets to each other and to the sun, reasoned that the whole solar system must have come from a nebula. It was a philosophical necessity that it should have such an origin. Herschel, the English astronomer, studied numerous nebulae thru his telescope and explained them as solar systems in the process of growth, and reasoned farther that our solar system must have had such a beginning. Laplace, the French mathematician and astronomer, arrived independently at the same conclusion. He worked out the explanation of a nebulous origin so much more completely than did any of his contemporaries that to most people the Laplacian hypothesis is *the* Nebular Hypothesis.

Please note the term nebular hypothesis. It is not a nebular theory. There are too many unknown things concerning nebulae and their transformation into solar systems to justify any other name than hypothesis, which is merely an intelligent or scientific guess.

Several different forms of a nebular hypothesis have been suggested, but because of the limits of this paper, two only will be discussed.

The Laplacian hypothesis: This hypothesis supposes that the solar system came from a gaseous nebula. If this nebula was spherical and of uniform density, its density was one-two hundred and fifty millionth of the density of the earth's atmosphere. This density can be computed by finding the density of a spherical body having the mass of the present solar system and the diameter of the orbit of the planet Neptune. It is impossible for most minds to conceive of such rareness. This nebula either from its beginning had a rotary motion or acquired it as it contracted.

The equatorial velocity of the nebula was great enough for a gaseous ring to form. This ring broke up and formed the outermost planet Neptune. Then more contraction of the nebula and an increase in the rate of rotation and another ring was thrown off. This ring formation continued until as many were thrown off as there are planets plus one extra which instead of forming into one mass broke up into many masses and formed the asteroids. The planets, at first in a gaseous condition, in turn threw off one or more rings and these became their satellites or moons. One exception in the formation of satellites is given for our own moon. It has been thought by some that it was thrown off as one great lump from the forming earth.

The planets were at first very highly heated. Much or nearly all of the present water mass, the hydrosphere, the present atmosphere, and much of the oxygen and carbon dioxide that have united with the rocks in the formation of oxides and carbonates, and the carbon dioxide that is represented in the coal formation were in the early atmosphere. This means that the atmosphere was very dense and that the pressure was many times that of the present atmosphere. Because of the large amount of the carbon dioxide and water vapor of this primitive atmosphere the earth cooled slowly and a crust was a long time in forming. The crust was very hot at first and was undoubtedly very uneven. If the crust was not of uniform density, and this is a fair supposition, as it cooled and contracted the denser material sunk the more rapidly and thus the oceanic depressions began. Water began to collect in the depressions when the crust was very hot. 500 degrees F. or hotter. This temperature was possible because of the great atmospheric pressure.

Some have modified this hypothesis by suggesting that a considerable amount of the gases, especially the water and carbon dioxide, were incorporated in the crustal rocks as they cooled.

For many years this hypothesis was accepted as the best and most plausible explanation of the earth's origin and few serious objections were urged against it, but as astronomy and geology advanced and applications were made to the hypothesis insurmountable obstacles appeared. The formation of rings when the nebula was so large and with so low a density and velocity is thought to have been impossible; and even if the rings were formed it is not at all understood how they could have been collected into a sphere. Again, the molecular velocity of gases was too great to hold the ring together.

It has been found that the satellites of Neptune and Uranus have a retrograde motion, that is, they revolve around their planets opposite to the direction of the planet's revolution. Phobos, the inner satellite of Mars, revolves in less than one-third the time of Mar's rotation. These, with some obstacles of a purely geological nature, seem to many persons to make the Laplacian hypothesis as commonly understood untenable.

It is easy to criticise this hypothesis but it is extremely difficult to suggest a better one. A new hypothesis to be of value must meet the objections to the old and also have fewer objections than the old.

The Planetesimal hypothesis: Dr. Chamberlin of the department of Geology and Dr. Moulton of the department of Astronomy of Chicago University have together worked out a new explanation known as the Planetesimal hypothesis. Outlines of this hypothesis have been published in various magazines, but the best explanation of it is to be found in Volume II of "Geology" by Profs. Chamberlin and Salisbury. It is from this book that a large part of this article is taken..

There are, it is estimated, 120,000 nebulae visible to the best telescopes and considerably over half of these are of the spiral type. If anything can be based upon the probabilities of the case it is more probable that our solar system came from a spiral than any other kind of a nebula; and if it can be shown that a solar system could evolve from such a nebula it is much more likely that the spiral nebula was the parent nebula.

It is untenable to suppose there was a real beginning of all things. If it were possible to account for the origin of the parent nebula its parent body must have come from another and this still from another, and thus it all seems to be an evolution of the cosmos, perhaps just a change of the form of the matter and the energy as they are understood to change upon the earth today.

The following is an attempt made to account for the origin of the spiral nebulae: Two great suns or systems in their movements thru space may have come sufficiently near each other to "disrupt" or pull each other or one only somewhat into pieces. The result of the disruption is the spiral nebula with two protuberances opposite each other. The disrupting body kept on its path thru space, perhaps never again to exert an appreciable influence upon the disrupted body. The chances of such a close approach are somewhat remote, but in the greatness of time it is neither impossible nor improbable. The disrupting body gave the direction of rotation to the nebula. The protuberances may be explained in the same way as the two high tides upon the opposite sides of the earth at one time.

An examination of the typical spiral nebula will show it to consist of a central mass which is to become the sun of the new system; of many "knots" which are to become the planets, of lesser knots to become the satellites and a large amount of diffuse nebulous material which is to be added, in a manner to be described later, to the central body and to the knots. The central mass and the knots and much of the diffuse material appear thru the spectroscope to consist of liquid and solid matter. The form of the nebula is sufficient to show that all particles are revolving in the same direction. The parent nebula of our solar system was very small as compared with some seen in the heavens.

According to this hypothesis the diffuse matter was added to the knots by a series of overtake collisions. Head-on collisions were impossible because all particles were revolving in the same direction. The various revolving particles are known as planetesimals and these may have been as small as molecules.

The revolving planetesimals had elliptical orbits and they must have had different velocities. It is possible and probable that one planetesimal was in its perihelion in nearly the same place that another was in its aphelion. The one in perihelion overtook the one in aphelion. It must not be understood that the overtake was just like one person overtaking another. It would be an overtake when the two bodies came near enough together that the larger would pull the other into it. It is said that the earth controls an area of 620,000 miles radius. The growing planets, of course, controlled smaller areas.

When the two bodies united a new orbit was formed which was the resultant of the orbits of the two bodies. The new body having a new orbit swept a little different part of space and overtook another and then there was another change of orbit. This continued thru all of the time the planet was growing and it is going on very slowly at the present time. As the orbits shifted the planets grew and came to occupy their present relations to each other.

The satellies grew as the planets grew, but independently of them. The retrograde motions mentioned above may be accounted for by the kind of collisions that occurred in the making of the body. It can be shown that the chances are much more in favor of a forward rotation than a retrograde rotation, and since the retrograde motion is an exception it is a point in favor of this hypothesis.

The planetesimal hypothesis explains the hypothetical stages that lead up to the known eras in a more satisfactory manner than any other.

The moon has no atmosphere. This has been variously explained. Some maintain that it did have one but it has all been absorbed within the rock mass of the moon, and that it is a condition that awaits our earth in the distant future. According to the planetesimal hypothesis this is incorrect. The moon does not have an atmosphere and never did have one because it is not large enough to hold one. Mars has an atmosphere but not as dense as that of the earth. His atmosphere is less because he is smaller than the earth and could not capture and hold as much as the earth.

The molecular velocity of the gases is so great that the attraction of the growing earth was not great enough to hold the gases until it was somewhere between the size of the moon and of Mars.

The atmosphere came from two sources: A part was captured in the ordinary overtake collision manner, and a part was brought to the earth in and with the larger captured planetesimals. In the earliest stages of the atmosphere there was no oxygen, and its presence is one of the most difficult parts of the hypothesis to explain. It has been suggested that it, the oxygen, came from oxides which were broken up by heat produced by the infall of the planetesimals. The heavier constituents of the atmosphere may have been relatively larger in amount in the beginning than at present.

There is a great difference between this and the Laplacian hypothesis in regard to the explanation of the temperature conditions within the earth and upon its surface during its development as well as at the present time. According to the Laplacian hypothesis the surface was hot in the beginning and a crust appeared only after a very long period of cooling. According to the plenetesimal hypothesis the surface has always been cool except locally where the heat was generated by infalls. This heat, in most cases at least, was soon lost by radiation.

The heat of the earth must have had an internal source. It is accounted for by a condensation of the central part, by a compression upon all parts due to growth in mass and to a molecular rearrangement of the interior rocks. The melting of the rock mass near the surface was due to an outward flow from the heated interior. This outward movement was aided by the attraction of the other masses upon the earth, causing a continual readjustment of the growing crust to the interior. In many places, perhaps over most of the earth, the melted rock reached the surface and melted it or covered it deeply. The older explanations considered the granite and other igneous rocks as found in oldest land areas as parts of the original crust.

Water must have been formed at an early stage upon the earth, but just at what time it is impossible to say. It is also hard to tell how it was formed; perhaps at the present stage of our knowledge we can't tell. It has been suggested that it was formed from various oxides which were broken up by the heat generated by the infalls, and the hydrogen of the atmosphere. The oxygen thus set free and which did not unite with hydrogen was added to the growing atmosphere as before mentioned.

The surface of the growing planet must have been very uneven, due to the method of growth. The water collected in the depressions which, of course, had no systematic distribution. As soon as the water began to percolate thru the rocks it dissolved parts of them and carried the soluble parts into the depositing basins. The parts dissolved were among the heavier elements. This would tend to make a difference in weight in different parts of the crust. The elevated parts were lighter and more siliceous in composition. The crust was continually readjusting itself to the interior and the heavier parts sank when there was a chance for sinking and the lighter parts were upheaved.

It must be remembered that the crust was at least 1,500 miles below the present crust when the water began to collect in these depressions or growing oceans, and that undoubtedly parts of the ocean were completely filled by planetesimals of some considerable size. The average depth of the ocean is about 11,500 feet and the average height of the land is 2,300 feet, an average difference of less than three miles between land height and ocean depth. If the crust has grown outward 1,500 miles or more since the oceans began the relative difference between the growth of the land and the ocean areas is very small. Perhaps the sinking of the ocean beds, due to increase in weight, would account for the whole difference.

A very great deal of water would be incorporated in the rock mass as the earth grew, and this water is now brought to the surface in small quantities thru volcanic action.

After countless ages the earth was fitted to receive life. It is not meant that conditions when life appeared were wholly as they are now. There must have been at atmosphere, oceans, light from the sun and a temperature not prohibitive.

Just how life appeared is a very delicate as well as difficult question to answer. Many people do not care much how the earth as a planetary mass was formed, any kind of an explanation will suit; but when an effort is made to account for life in the same rational manner as other things are accounted for the scientist is told that he is treading upon forbidden ground, ground that is already preempted and that no trespassing is allowed.

How did life begin upon the earth? Was it brought on an ether wave from some far-away world and planted upon this planet as some have held? Extremely unlikely. Life must have begun here. Protoplasm, a substance found in all living things contains the elements, carbon, hydrogen, oxygen, nitrogen and usually sulphur and phosphorus. Meteorites have been carefully studied and it is known that some of them contain unstable compounds with all or some of the elements necessary to protoplasm. Might it not be possible that a meteorite, or several of them, and this but another name for a planetesimal, struck the earth along the seashore? Here the water of the loose rock with salts in solution and the unstable compounds of the meteorite set free and changed by the heat of the infall formed a new chemical compound and which was the ancestral compound from which the protoplasm of today came. This is but an attempt to account for life in a logical manner.

The first living thing must have been more plant than animal because all food in the beginning was of an inorganic nature.

Because life began in this way it does not mean that we know what life is. It is just as inexplicable as it ever was. We can't say that life is or is not the result of the union of a few elements; but this much seems to be well established: a few elements only are necessary to form protoplasm. "No finite mind knowing what it does of the elements of a compound can predict the properties of the compound."

In the above there is an attempt to account for the earth up to the time it became habitable and conditions upon it were something like the present. In its subsequent history almost as great, if not just as great, problems must be solved.

Life starting from an extremely simple beginning differentiated at first into plants and animals and then into several types of each, some existing today and others long ago extinct. Why should not all have died or why should any have died?

Geological history may be divided as human history is divided. The hypothetical part, the traditional part, the part in which a few authenticated events are recorded and the true or more authenticated history. The history is a long one and even when the geologist is surest of his data he knows very little about the great problems that most interest him.

How to explain an almost tropical climate within the polar areas during the long polar nights, and glaciation of great magnitude on the borders of and within the Torrid zone are two problems. Sediments, thousands of feet in thickness, found in the mountains, show great changes. What caused the changes?

The earth is not a completed planet. Changes are going on today as much as ever. Living things are changing continually. If we cannot explain the changes that occur about us every day how can we hope to explain the changes that occurred in the millions of years that have passed? We have unmistakable evidence of the changes of the past and can hope to explain them only as we study and understand the changes of the present.

The long history of the earth, the slow struggle upward of life, one branch of which culminated in man, is a great prophecy of what we may become in that future when the present problems will be solved and others more intricate will be pressing for solution.

DESIGN OF A RAILWAY PLATE DECK GIRDER BRIDGE.

BY ARTHUR D. BUZBY, C. E. 1911.

Plate girders are now considered to be the most reliable and durable type of metal bridge, the majority of railroads using them exclusively in spans up to 30 feet. Deck girders are in wide general use in spans up to 100 feet, with scattered instances of considerably longer spans. The girder is what is known as a built up I beam, consisting of channels and angles, angles, angles and plates, riveted tc a web plate at top and bottom to form the flanges.

DATA.

For the design herewith submitted, the following data is assumed:

Span, 86′-6″ out to out (84′-0″ C to C bearings).

Loading—Cooper's Class E 50.

Specifications—American Railway Engineering and Maintenance of Way Association, 1910.

Width—C to C girders—7′-0″.

Ties—8″ × 8″ × 10′-0″, spaced 14″ C to C.

Weight of timber taken at 50 lbs. per cu. ft.

Dead Load $= L(124 + 10L)$; where $L =$ span in ft.; weight of tracks, fastenings, etc., is to be added to the result given by this formula, and the sum to be considered total dead load per lineal ft. of bridge.

DEAD LOAD.

D. L. $= 84(124 + 10 \times 84) = 81000\#$ or 963# per lin. ft. of bridge.

Rails and fastenings (from Spec.) 150# lin. ft. of bridge.

Ties 8″ × 8″ × 10′-0″ $= 8 \times 8 \times 120 \times 67 \times 50 \div 12 \times 12 \times 12 \times 84 =$ 177# lineal ft, of bridge.

2 Steel guard rails at 80# yd. = 53# lin. ft. of bridge.

2 Wooden guard rails 8″ × 5″; 2 × 8″ × 5″ × 12″ × 50 ÷ 1728 = 27.8# lin. ft. of bridge.

Total, 1370.8# lin. ft. of bridge, or 1/2 × 1370.8 = 685.4# lin. ft. of one girder.

Timber at 50# cu. ft.

Ties spaced 14″ C to C = 67 requ'd. W = 685# lin. ft.—Equivalent Uniform Dead Load.

MAXIMUM LIVE LOAD MOMENT.

$Wl \div Wt = \frac{1}{2}$ (Criterion). The w't. of the train on the bridge up to the panel point in question or to the center of the bridge divided by the total train load on the bridge should be nearly equal to ½.

From Moment Table Page 87.

Try wheel 12 at the middle. $M_{max} = ((Ma \div 2) - Ml) \div 2 \times 5/4$ for $E50 = W''l^2 \div 8$ (See Pratt Analysis by S. B. Ehrenrich).

$204 \div 388 + 5 \times 4 = 204 \div 408 = \frac{1}{2}$ wheel 12 gives max. moment.

Ma = moment to left about right reaction.

Ml = moment of load to left of point in question about that point.

$Ma = Pg + Px + Wx^2 \div 2$, where g is dist. from C to C of wheel loads to the beginning of the uniform train load. X is length of uniform train load. W is wt. of of uniform train load per lin. ft.

$Ma = (15588 + 388 \times 7) + (4 \times 7^2) \div 2 = 18402$ kip ft.

$M_1 = 3476$ kip ft.

$M_{max} = \frac{1}{2}((18402 \div 2) - 3476) \times 5/4 = 3578$ Kip ft. $= w''l^2/8$, $w'' = 4.056$ Kip ft.

$W'' = 4056$ lbs. per lin. ft.—Equivalent uniform live load.

MAXIMUM LIVE LOAD SHEAR.

Run on the biggest load which can come on the bridge, and taking moments about the right reaction, determine the left one. The largest load will be on the bridge when wheel 15 is 80′ from the left end, with wheel 2 on the left abutment..

$R_1 l = P \times 84$.

$R_1 = (19872 + 4 \times 470)/84 \times 5/4 = 324000\#$ Max. L. L. Shear for both sides of bridge.

Max. L. L. Shear for 1 girder $= 324000/2 = 162000\#$ for 1 girder.

$162000 = wl/2$, or $w^1 = 3854\#$ per lin. ft. —Equivalent Uniform Load.

LIVE LOAD SHEARS.

The shears will now be figured from the equivalent uniform loads at points 0′ — 0″, 17′ — 0″, 42′ — 0″, from the left C of bearing, which are the points where the web splices occur, as determined later. For location of points see stress sheet.

Live Load Shear at $A = 3854 \times 84^2 \div 2 \times 84 = 162{,}000\#$.

Live Load Shear at $B = 3854 \times 67^2 \div 2 \times 84 = 87{,}400\#$.

Live Load Shear at $C = 3854 \times 42^2 \div 2 \times 84 = 37{,}600\#$.

To each of the stresses thus far computed must be added the Impact Stress as determined by the formula given in specification, par. 9.

$I = S\ 300/L + 300$.

Summary of Stresses.

Max. L. L. Moment ....3,578,000 ft. lbs.
Impact Moment $= 3{,}578{,}000 \times 300 \div 84 + 300 =$ ......2,800,000 ft. lbs.
Dead Load Moment $= 685 \times 84 \times 84 \div 8 =$........... 604,000 ft. lbs.
Total Max. Moment ....6,982,000 ft. lbs.
(A) Live Load End Shear ....162,000#
Impact Shear $162{,}000 \times 300 \div 84 + 300$...................126,500#
Dead Load Shear $685 \times 84/2$ .... 28,800#

Total End Shear ....317,300#
(B) Live Load Shear at B.................................... 84,400#
Impact Shear at B $84{,}400 \times 300 \div 67 + 300$................68,600#
Dead Load Shear at B, $685 \times 67/2$.........................22,900#

Total Shear at B ....175,900#

(C))Live Load Shear at C.....................................37,600#
Impact Shear at C, $37{,}600 \times 300 \div 42 + 300$................32,900#
Dead Load Shear at C, $685 \times 42/_2$.......................14,400#

Total Shear at C....................................84,900#

Adding the equivalent loads together, the uniform dead load, the live load and the impact, we get the total equivalent uniform load for Max. Moment $= W + W'' + W'''$.

$W = 685$# lin. ft.
$W'' = 4056$# lin. ft.
$W''' = 3174$# lin. ft.

$W'''' = 7915$# lin. ft.

M total $= W'''' L^2/8$, or $W'''' = 6982000 \times 8 \div 84 \times 84 = 7916$#.
Total Moment at $B = 17 \times 7916 (84 - 17) \div 2 = 4{,}508{,}000$ ft lbs.
Total Moment at $C = 42 \times 7916 (84 - 42) \div 2 = 6{,}981{,}900$ ft lbs.

Web...The depth of the web should be assumed at about 1/10 of the span, and the required web area will be the total max. shear divided by the max. allowable unit stress per sq. inch—

Area. $317{,}300/10{,}000 = 31.73$ sq. inches, and using a 90″ plate the req'd. thickness will be $31.73/90 = .330$ inches; therefore use a web plate $90'' \times 3/8''$.

Flanges. According to spec. par. 29, 1/8 of the gross web area may be regarded as flange area. Assuming the center of gravity of the flange section to be 1″ from back of angles, the approximate effective depth = depth of web plate plus clearance at each end less the distances of the center of gravity of each flange from the back of the angles—

Effective depth $= 90'' + (2 \times 1/8) - (2 \times 1'') = 88.25''$.

Then the stress on the flange = max. moment in inch lbs. ÷ eff. depth in inches or Flange stress = 6,982,000 × 12 ÷ 88.25 = 950,000#.

Flange area req'd = flange stress ÷ allowable unit stress = 950,000 ÷ 16,000 = 59.37 sq. in.

The following flange section will be tried:

	SQ. IN. GROSS		SQ. IN. NET
$\frac{1}{8}$ web=$\frac{1}{8}\times\frac{3}{8}\times90$	= 4.22		4.22
2Ls, $8\times8\times\frac{3}{4}$	= 22.88	22.88—$(2\times2\times1\times\frac{3}{4})$	18.88
2 plates $18\times\frac{3}{4}$	= 27.00	27—$(2\times1\times\frac{3}{4})2$	24.00
1 plate $18\times\frac{11}{16}$	=12.375	12.375 −$(2\times1\times\frac{11}{16}$	11.40
Total gross area=66.485		Total net area	58.50

Calculated Center of Gravity of Flange Section.

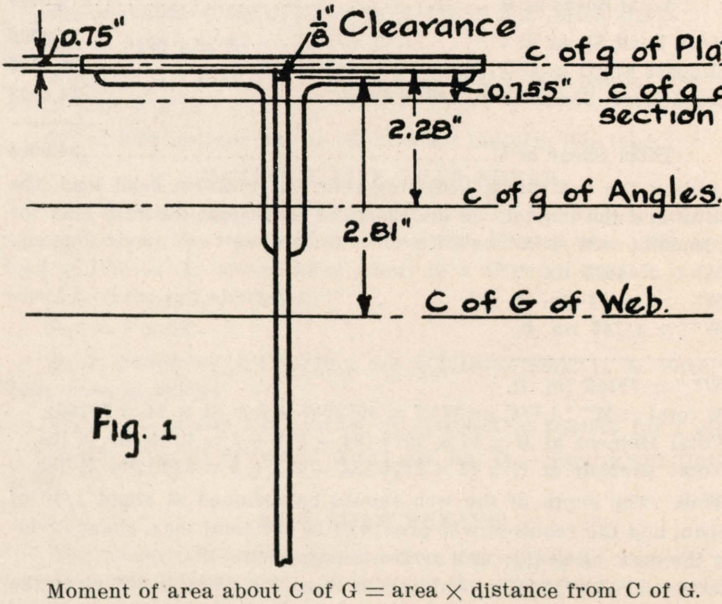


Fig. 1

Moment of area about C of G = area × distance from C of G.

(2 × 11.44) × (2.28 + 0.375) = 60.632 inches 3rd = moment of Ls.

4.22/2 × 2.81 = 5.93 inches 3rd Mom. of Web plate.

60.632 + 5.93 = 66.56 inches 3rd Total Mom. of section.

66.56/58.50 = 1.13″ dist. C. of G. of section from c of g of cover plate.

1.13 — 0.375 = 0.755″ dist. of C. of G. of section from back of angles.

The true effective depth = 90 + 0.25 — (2 × 0.755) = 88.74 inches at end of bridge.

The actual flange stress is 6,982,000 × 12/88.74 = 945,000# and the true required area = 945,000/16000 = 59.06 sq. in. so the assumed flange section will be sufficient as it comes within the allowable limit of 3% difference from the required area.

Effective Depth at Middle of Bridge.

Moment of Ls=(22.88) (2.88+1.5+-.34375)=94.357 in. 3rd.
Moment ½ Web=4.22/2×(2.81+1/8+1.5+.3437) =10.08 in. 3rd.
Moment Plates=(27.00) (0.75+0.3437) =29.538 in. 3rd.

Total Moment 133.975 in. 3rd.

133.975/58.50 = 2.29 inches, dist. C. of G. from top plate.

2.29 − (1.50 + .34375) = 0.446″ dist of C. of G. of section from back of angles.

True effective depth at middle = 90 + .25 − 2 × 0.466 = 89.318 in.

Average eff. depth from end to middle = (88.74 + 89.318)/2 = 89.-029″.

Length of Flange Plates.

These lengths may be determined graphically by plotting a curve of the maximum moments. These moments and the flange area are directly proportional, each member of the flange section taking its proportional moment.

Total net area 58.50 sq. in. = 6,982,000 ft. lbs. total max. moment.

Moment taken by web = 4.22/58.50 × 6,982,000 = 504,500 ft. lbs.

Moment taken by two angles = 18.88/58.50 × 6,982,000 = 2,250,000 ft lbs.

Moment taken by 3 plates = 35.40/58.50 × 6,982,000 = 4,226,000 ft. lbs.

Total plate area = 35.40 sq. in.

Then 2 — ¾″ plates each take 13.5/35.4 × 4,226,000 =1,430,000 ft. lbs.

1 —11/16″ plate takes 12.375/35.4 ×4,226,000 = 1,362,000 ft. lbs.

To the same vertical scale as the max. moments were plotted, and on the same diagram, plot the moments taken by each plate; project these points horizontally until they intersect the max. moment curve, and the required length of plates may then be scaled off directly to the same horizontal scale as was used to make the drawing. The top flange plate next to the angles is run the entire length of the bridge to cover the flange angles and to stiffen them near the ends.

Scaling the plates from the diagram, we get—

18″ × ¾″ .... 71′ — 6″, use 86′ — 6″
18″ + ¾″ ....58′ — 0″, use 60′ — 0″
18″ × 11/16″ ....43′ — 6″, use 45′ — 6″

Flange Riveting.

The horizontal increment of stress at any point may be obtained from the formula:

(A) Hor. stress $= Sx/d \times Af \div (Af - Aw/_8)$ (Morris) where Sx is the max. shear at the point in question, Af = area flange at that point, and Aw = web area.

Hor. Increment at $A = 317{,}300/88.74 \times 35.255/35.255 + 4.22 =$ 3210# per lin. inch.

Gross areas are used in this determination. The effective depth varies from the end of the bridge to the middle, reaching a maximum there. For intermediate points the average of these two may be used.

The vertical load from the floor is 203.9 lbs. per lin. ft; and the flange weight at the end is 114# per lin. ft., making a total:

Dead load of 318# lin. ft, or = ...	26.5 lbs. per lin. inch
Live load (from spec. par. 7) 25000/42 = ...	595.0 lbs. per lin. inch
Impact (from spec. par. 9) 100% = ...	595.0 lbs. per lin. inch
Total vertical load = ...	1,216.5 lbs. per lin. inch

Resultant Stress on Rivets $= \sqrt{(3210)^2 + (1216)^2} = 3320$ lbs. per lin. inch.

Required rivet pitch $= 6013/3320 = 1.81''$ where 6013 is single shear value of a ⅞″ rivet, unit stress of 10,000#/sq. inch.

In finding the Hor. increment at B, use the average effective depth as found p 37. The vert. load will here be increased to 1220 lbs. per lin. inch.

(B) Hor. increment at $B = 175{,}900/89.03 = 1970$#/lin. in.

Vertical Load = 1220#/lin. in.

Resultant stress $\sqrt{(1220)^2 + (1970)^2} = 2312$ lbs. per lin. inch.

Requ'd. rivet pitch $6013/2312 = 2.6$ inches.

Pt. C. in finding the horiz. increment here, use the eff. depth at the middle.

(C) Hor. increment at C. $= 84{,}900/89.32 = 950$ lbs. per lin. in.

Vertical Load = 1223 lbs. per lin. in.

Resultant Stress $\sqrt{(1223)^2 + (950)^2} = 1550$ lbs. per lin. in.

Requ'd rivet pitch $6013/1550 = 3.18$ inches.

These rivet pitches are now plotted and the pitches actually used must come within the curve.

Flange Splices.

As 8″ × 8″ × ¾″Ls can be obtained in lengths exceeding 86′ — 6″, no flange splices are necessary.

Stiffeners.

The stiffeners are designed according to specifications No. 16 and No. 79.

End Stiffeners: The end shear is 317,300# and the value of a 7/8″ rivet in single shear is 6013; therefore the req'd. No. rivets = 317300 ÷ 6013 = 53. This number can be put in 2 pairs of stiffener angles with a single row of rivets in each angle. The stress on each pair of Ls = 317300 ÷ 2 = 158650#. For the required Ls try 2 Ls 7″ × 3½ × ½″.

The allowed unit stress from spec. 16, = 16000 — 701 ÷ r.
16000 — 70 × 88 ÷ 4.52 = 14640 lbs. per sq. inch.

The requ'd. area of 1 pair of Ls = 159650 ÷ 14640 = 10.83 sq. inches. We can use therefore 2 Ls 7″ × 3½″ × 9/16, whose area is 11.18 sq. inches.

Intermediate Stiffeners.

From spec. 79, minimum size of outstanding leg of Ls is 90/30 + 2 = 5 inches. Therefore use 2 Ls 5″ × 3½ × ⅜″.

From the spec. the minimum spacing between stiffeners d = t ÷ 40 (12000 — s) where d = clear distance between the stiffeners of flange Ls, t = thickness of web, s = shear per sq. inch.

Required spacing at A = (⅜″ ÷ 40) (12000 — 317300 ÷ 34) = 67 inches.

Required spacing at B = (⅜ ÷ 40) × (12000 — 175900 ÷ 34) = 171 inches.

Required spacing at C = (⅜ ÷ 40(× (1200 — 84900 ÷ 34) = 237 inches.

Web Splices.

The total length of the girder is 86′ —6″. Plates 90″ × 3-8″ are listed in the Cambria handbook up to 300″ or 25′ long. Therefore the web must be spliced at 3 points, 2 occuring at 17′ — 0″ and one 42 — 0″ from the C of bearing. The total moments at these points must be resisted by two small splice plates, one at the top flange and one at the bottom, of an assumed width of 9″. A clearance of 1/8″ should be left between the edges of the plates and angles. The moment splice plates will be designed for the point C as the greatest moment occurs there and these splice plates will be used at the other splice points.

The actual flange area effective at point C is 58.50 sq. in. The total moment at this point is 6,982,000 ft. lbs. and therefore the bending moment taken by the web = (4.22 ÷ 58.50) × 6,892,000 = 504,000 ft. lbs. The 9″ splice plates must resist this moment, and the stress in them is 504,000 ÷ 65″ = 93,100 lbs., where 65 inches is distance C to C of the two plates.

The maximum allowable unit stress on the extreme fiber of the girder is 16,000# sq. in., and the maximum allowed unit stress on the splice plates will be proportional to their distances from the neutral axis of the girder, or (32.5 ÷ 46.66) × 16,000 = 11,650#/sq. in. where 32.5 is dist. from center of plate to neutral axis of girder; and 44.66 is half the effective depth at the center. Requ'd. area in splice plates = 93,100 ÷ 11,650 = 8.00 sq. in.

This will require 2 plates 12″ × ½″, net area = 12 − (2 × 4 × 1 × ½) = 8.00 sq. in.

The number of rivets req'd. on each side of the splice = 93,100 ÷ 6,563 = 14 using the value of ⅞″ rivet in single shear on a ⅜ plate.

The shear splice plate will be designed for point B, as the shear here is greater than at the middle, and the plate so determined will be used for all the splices.

Total max. shear at B = 175,900#.

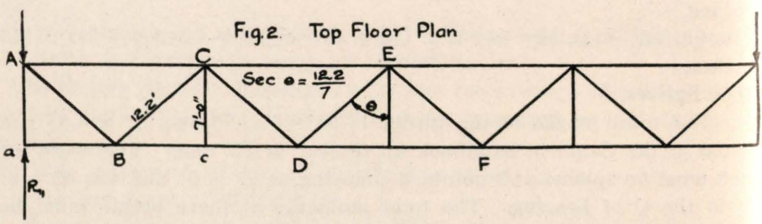
Req'd. No. rivets = 175,900 ÷ 6,563 = 27 rivets on each side of splice.

Lateral Bracing.

This bridge will be designed for two lateral systems, one in the plane of each flange. The span will be divided into 8 panels; 6 of 10 ft. each, and 2 end panels of 12 ft. each. Cross frames and stiffeners will be put in at every second panel point.

From spec. par. 10, panel load = 200 + 200 + 10% of 5,000 = 900#/lin. ft. of girder, to be considered as moving load.

TOP FLOOR PLAN.



The stresses in the web members will be solved by the method of coefficients as explained in Art. 7, Chap. 21, Malcolm's Graphic Statics.

The criterion for maximum live load web stresses, is to load the longer segment of the truss only.

Then the live load coefficient in any web member = ½m (m + 1) /N, where m = number of panel loads on the truss for maximum stress. n = number of panels in the truss.

Diagonals.

Live Load coeff. for member $AB=((7+1)\div2)\frac{7}{8}=\frac{28}{8}$
" " " " " $BC=((6+1)\div2)\frac{6}{8}=\frac{21}{8}$
" " " " " $CD=((5+1)\div2)\frac{5}{8}=\frac{15}{8}$
" " " " " $DE=((4+1)\div2)\frac{4}{8}=\frac{10}{8}$

Verticals.

Live Load coeff. for member $Aa=((8+1)\div2)\frac{8}{8}=\frac{36}{8}$
" " " " " $Cc=((6+1)\div2)\frac{6}{8}=\frac{21}{8}$
" " " " " $Ee=((4+1)\div2)\frac{4}{8}=\frac{10}{8}$

The stress in diagonal members = panel load × coefficient × length of member/depth of truss.

Panel load = 900 × 84/8 = 9450 lbs. approximately.

Stress in AB = 9450 × 28/8 × 12.2/7 = ... 57,600 lbs.
Stress in BC = 9450 × 21/8 × 12.2/7 = ... 43,200 lbs.
Stress in CD = 9450 × 15/8 × 12.2/7 = ... 30,800 lbs.
Stress in DE = 9450 × 10/8 × 12.2/7 = ... 20,600 lbs.
Stress in Aa = 9450/2 × 36/8 × 1 .... 21,200 lbs.
Stress in Cc = 9450/2 × 21/8 × 1 .... 12,400 lbs.
Stress in Ee × 9450/2 × 10/8 = 1 .... 5,800 lbs.

From spec. par. 74, minimum angle to be used is 3½″ × 3″ × 3/8″.

The unsupported length may be taken as the distance between edges of flange angles = 12.2′ − 2 × 14.125″ = 9.66 ft. = 116 in.

For AB.

Try 1L 6″ × 6″ × ½, area 5.75 sq. in. Least radius of gyration is 1.19. Then allowed stress = 16,000 − 70 L/r = 16,000 − 70 (116/1.19) = 9180 lbs. per sq. in.

Then angle 6″ × 6″ × ½″, is worth 5.75 × 9180 = 52,780 which is too small. So try 1L 6″ × 6″ × 9/16″, area 6.44 sq. in., least radius of gyration 1.18.

Allowed stress = 9120 lbs. per sq. in.

Then L 6″ × 6″ × 9/16″ is worth 9120 × 6.44 = 58,732 lbs. and there fore angle is OK for AB.

For BC.

Try 1L 6″ × 6″ × 7/16″, area 5.06 sq. in., least radius of gyration 1.19.

Allowed unit stress = 9180 lbs. per sq. inch.

Allowed total stress = 9180 × 5.06 = 46,458 lbs. Therefore 1L 6″ × 6″ × 7/16″ is OK.

For CD.

Try 1L 6″ × 6″ × 3/8 , area 4.36 sq. in., least radius of gyration = 1.18.

Allowed unit stress = 9,120 lbs. per sq. in.

Allowed total stress = 9120 × 4.36 = 39,673 lbs.

Therefore 1L 6″ × 6″ × 3/8 is OK.

For DE.

Try 1L 5″ × 5″ × 3/8″, area 3.61 sq in., least radus of gyraton = 0.98.

Allowed unit stress = 8040 lbs. per sq. in.

Allowed total stress = 8040 × 3.61 = 25000 lbs.

Therefore 1L 5 × 5 × 3/8 is OK.

For Aa.

Use 1L 5″ × 5″ × ⅜″, same as for DE.

For Cc.

Use 1L 5″ × 5″ × 3/8″, and for all verticals use this angle.

For Ee.

Use 1L 5″ × 5″ × 3/8″.

For bottom laterals use Ls 4″ × 4″ × 3/8″.

End Cross Frames.

These must be designed to carry all the wind load to the abutment assuming that half the load is transmitted by each angle, to the supports, one in tension and the other in compression.

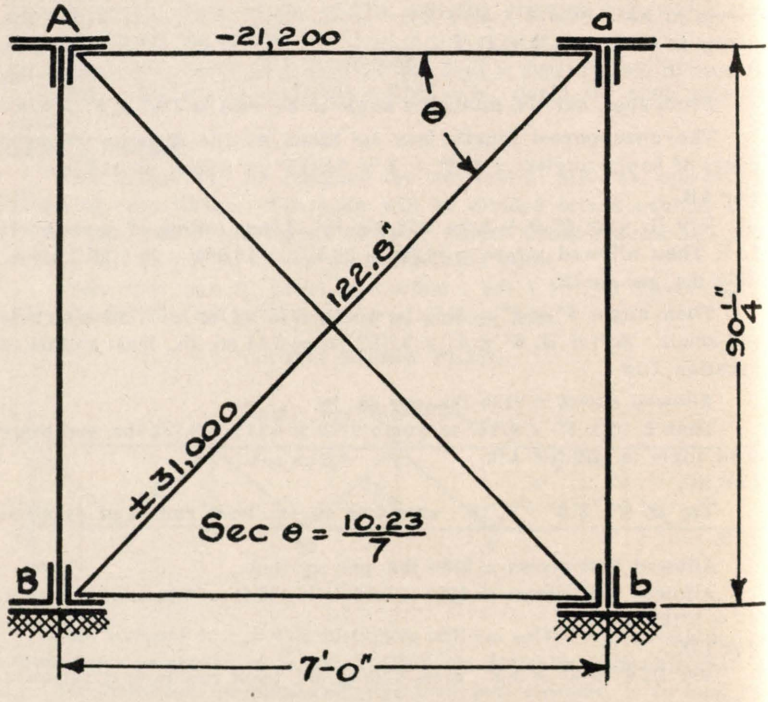


Fig. 3 End Cross Frame.

Stress in Strut Aa = 21,200#.

Stress in Diagonal Ba = 21,200 × sec. O = 31,000#.

If the angle tends to bend outward the length is 122.8″, but if it bends in a plane at right angles to that, since the angles are fastened at the middle, the length will be only 122.8/2 = 61.4″. Therefore it is best to use an angle with unequal legs, the larger one extending outward. This also allows the greatest rectangular radius of gyration to be used.

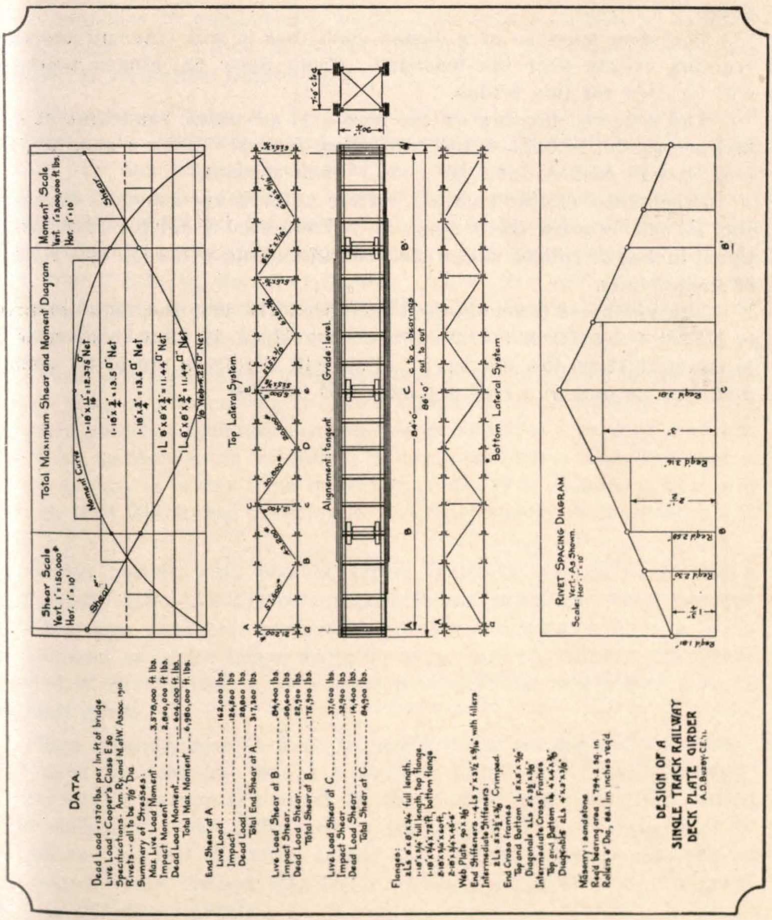
Try 1L 5″ × 3½″ × 3/8″, area 3.05 sq. in., greatest radius of gyration 1.61.

Allowed unit stress = 16000 − 70 (122.8/1.61) = 10,660 lbs..

Req'd area = 31,000/10,660 =2.91 sq. in. Therefore use for diagonals 1L 5″ × 3½″ × 3/8″.

Intermediate Cross Frames.

These are usually not designed, since the stresses which come upon them are not easily ascertained. Good practice says that for spans from 65 to 110 ft. a 4″ × 4″ × 3/8″ angle with 4 rivets used at



connections, will be safe. Therefore for all intermediate stiffeners we will use Ls 4″ × 4″ × 3/8″.

The number of rivets necessary for the end connections of the lateral is determined by the value of a rivet in single shear.

AB requires 57,600/6013 = 10 rivets.

BC requires 43,200/6013 = 7 rivets.

CD requires 30,800/6013 = 5 rivets.

DE requires 20,600/6013 = 4 rivets.

End cross frame struts Aa require 21,200/6013 = 4 rivets.

End cross frame diagonals require 31,000/6013 = 5 rivets.

The intermediate cross frame diagonals will have 4 rivets in the connection.

Shoe.

The shoe must be of a design such that it will transmit the end reaction evenly over the masonry. From Spec. 63, hinged bolsters will be used for this bridge.

The required bearing on the masonry, assuming sandstone at 400 lbs. per sq. in. is 317,300/400 = 794.2 sq. inches. Using a shoe $2\frac{1}{2}$ ft. long, it will require 794.2/30″ = 26.5″ inches width.

From Spec. 62, the smallest rollers allowed are 6 inches diameter and it will require from Spec. 19 317,300/600d = 317,300/3600 = 88.1 lineal inches of rollers under each bearing. This will require 4 rollers 22 inches long.

The pin must transmit the shear properly, and the required area is $317300 \div 2 \times 12000 = 13.2$ sq. in. From Spec. 18, unit shear of pins is taken at 12000 lbs. per sq. in. This will require a pin of 4.1 inches diameter, so pins of 4 1/8″ diameter will be used.

POSSIBILITIES IN IRRIGATION ENGINEERING.

N. E. WOFARD, B. C. E.

UNDER the head of Civil Engineering are comprised several distinct branches of engineering work. One branch, which is comparatively new, but which is rapidly coming to be a very important branch, is Irrigation Engineering.

Since the passing of the Reclamation Law and the wide-spread discussion in periodicals and magazines of "Reclaiming the Arid West," the Easterner has become more or less familiar with irrigation and its possible resources. However, to understand the real value of irrigation and the ingenuity required to execute the work, one must cross the great arid plains and peer into the precipitous ravines and inky depths of the river canons. There are vast areas of rich, tillable land throughout the Rocky Mountain States, which but need an ingenious hand to convert them into so many Edens.

Three-score years ago irrigation was almost unknown in the region where it is now attracting so much attention. Perchance a few progressive pioneers bethought themselves of artificial means of moistening their garden spots by taking a spade or shovel and diverting a small stream of water from a nearby creek. The meadows were all bottom land which was drenched by the flood waters in the early summer.

When bottom land became scarce, settlers learned that with a little work, uplands could be turned into hay meadows. Each rancher was his own engineer, however, "running the ditches by eye." As the demand came for larger flows of water thru the ditches, the above method of survey had to give place to one which would give a more uniform grade.

Two leveling devices were in common use among early ditchers. The most common one was a sixteen or twenty-foot "straight edge" supporting a carpenter's level in the middle and with legs nailed onto each end. The difference in the length of legs was equal to the amount of fall the ditch should have in the distance spanned by the board. This method was fairly reliable and is still used a great deal in running laterals.

The other method consisted of an A whose cross bar was graduated to tenths of a foot and from the apex of which was suspended a heavy plumb bob. The contrivance was constructed of light material and made to span ten or twelve feet. It was manipulated in the same manner as a pair of dividers when spacing a line. The grade of the ditch was determined by the distance of the plumb line from the center of the cross bar.

Bye and bye some engineer appears on the field of action with one of those wonderful instruments and advertises as a Ditch Surveyor: he was the forerunner of the modern Irrigation Engineer.

As yet, there was no system to the diversion and appropriation of waters. Disputes arose which were frequently settled by the "armstrong" method, or with revolvers, there being no laws by which to adjust matters.

It soon became apparent that "Water being essential to industrial prosperity, of limited amount and easy of diversion from its natural channels, its control must be in the state, which, in providing for its use, shall equally guard all of the various interests involved." (Water law of Wyoming, Art. 1, Sec. 31.)

As might be expected, in infancy the profession of Irrigation Engineering was abused by incompetence. Maps were sent in to the State Engineer which did not accurately show ditch or stream location or mistakes were made in township or section locations of lands to be irrigated. Plats were even sent in representing projects on which there had never been an instrument set up. Endless confusion and trouble, and expense as well, were the results of this slipshod work.

As a remedy for this evil, many of the states require an engineer to pass an examination,, which secures for him a state license, before any of his work will be accepted at the office of the State Engineer. It is obvious that these requirements have lifted the profession to a higher plane, encouraged a higher degree of efficiency in the engineers, and guarded the public against the careless or incompetent man.

I have thus far endeavored to give the evolution of the Irrigation Engineer and his profession up to the present time. Now we will consider the possibilities the future holds in store for him.

As previously stated, the early settlers located on the river bottoms. As immigration to the west increased, the uplands and mesas which were easiest of access to water were dotted by homsteaders' cabins, and were gradually changed from the desert hue to verdant splendor. Now most of the tillable land adjacent to the streams and rivers, which traverse the arid tracts, is reclaimed; so also are most of the flowing waters of said streams appropriated. Hence it is obvious that irrigation in the future will be attended with more difficulties and consequently with more expense than has been met with in the past.

Future reclaimants must look to reservoir storage for their water supply. The location of these reservoirs ofttimes presents no small problem to the engineer in charge. If the reservoir is to be of any considerable size, the geological formation and phenomena of the proposed site must be investigated to ascertain whether or not it would hold water with no great percent of leakage. There is also great responsibility on the shoulders of the engineer who undertakes to construct a dam to hold back hundreds, and in some cases thousands, of acre feet of water. Most reservoir sites are located on natural water courses on whose banks are thousands of dollars worth of property. If the dam construction is not absolutely safe, all property lying below it, as well as the lives of the property owners, are endangered.

Many of the best reservoir sites being canons lying deep in the mountains, and it also being necessary to head a canal, of any considerable size in or hard by the mountains in order to cover the plateaus and uplands, very perplexing problems are met with in the building of the canals.

It is generally conceded that civil engineers, in any of the component branches comprising the profession, must be hardy, adventurous, trifling with hardships, and, in a sense, unsocial beings. To whatever degree this may be true in other branches of the profession, it is most certainly true of the branch under discussion. This assertion will not in the least be disputed by anyone who has experienced the adventurous thrill of mountain climbing or canon exploration. Ofttimes when the irrigation engineer bids his wife and family, if perchance he has one, a fond farewell, it is with the feeling that it may be for the last time.

The reader will be convinced of the truth of this statement when he reads the thrilling adventures of the reclamation engineers who explored the Gunnison Canon, as hereinafter related.

Another method of irrigation is now undergoing experiment in several localities. This method consists of wells and electric pumps as a means of drenching the thirsty soil. Most of the mountain streams are rich in water power, and it is believed that in many places the construction and maintenance of a power plant and electric pumps would prove more economical and equally as efficient as the construction and maintenance of a canal and lateral system.

In case of a triumph of this system, as in all probability there will be, the irrigation engineer must needs increase his magnetic powers.

To bring before the reader the true nature of irrigation reconnaissance, I can do no better than to relate the actual experience of engineers who have been over the ground.

Few men live to tell a tale such as W. W. Torrence and A. L. Fellows, engineers of the Reclamation Service, told to the world twelve years ago. Perhaps no one will ever tell such a story again, and had these bold engineers known the horors which awaited them in those inky, rumbling depths of the Black Canon, they would not now be the heroes of a story thrilling as that of the Alpine climber.

Where the Gunnison river madly courses down the western slope of the Colorado Rockies, it is encased in deep, rugged ravines. The famous Black Canon for years bade defiance to the boldest explorers. Gunnison, who explored the river pronounced the canon impenetrable, as did also Prof. Hayden of the geological survey. Geologists who had been lowered a thousand feet into the canon declared that no human could go farther and live. The yawning depths seemed to be nothing less than a byway to Pluto's own domain.

A little French settler, named Luzon, as he sat in his little cabin surveying his verdant forty acres and contrasting it with the surrounding desert of the Uncompahgre Valley, first conceived the idea of turning the distant rumbling Gunnison from its course to convert the Uncompahgre Valley into a garden spot.

One morning the following telegram was received at the Reclamation headquarters, at Washington:

"Can the Gunnison river be made to water the Uncompahgre Valley?"

A. L. Fellows received the telegram, read and re-read it, then handed it to W. W. Torrence. Never was such dangerous undertaking proposed by the bureau. To enter the canon at the place where it was surmised the tunnel might be built was out of the question, for the strongest silk cord would be chafed in two in scraping over the sharp cliffs, and the explorer would plunge headlong into eternity.

The only way to reach the point in question was to enter the canon fourteen miles up stream, where in a wall fifteen hundred feet high a single vulnerable point presented itself, and to follow the course of the turbulent stream. When once one entered the canon, he would be obliged to pursue it to the bitter end, for retreat against such a current could never be accomplished.

However, Mr. Torrence was bold enough to pilot a party of four sturdy, daring engineers, who, after bidding farewell to a group of friends on the chasm's brink, were lowered by ropes into the roaring gloom below.

To those who have never picked their way along a roaring, foaming torrent, or clutched the crevices of a perpendicular cliff, picking their way where the slip of a foot, or giving away of a stone meant instant death—I say to such the story which follows will not be manifest in its most terrorizing aspects. It took iron will as well as iron nerves to undertake such an exploit as did Mr. Torrence and his party.

The party equipment consisted of stout, oak frame boats, covered with canvas, tinned foods and hardtack to last a month, surveying instruments, cameras, and notebooks, all protected in tin boxes.

With volleys from revolvers they signaled to the friends above that the exploration had begun. It was agreed that parties would be stationed along the banks above and report to the anxious friends and relatives the progress of the exploring party from day to day.

From the start the men had to struggle through drenching icy spray, caused by the lashing of the torrent against boulders, and over glassy rocks, holding fast all the time to their boats, which on being loosed would have shot down stream like arrows. In places, rapids only ankle deep almost swept the men from their feet. They tied themselves to a common cord and with boats and provisions on their shoulders advanced very cautiously.

Hardship and privation were gnawing at the vitals of the men. The bottom of the cannon grew pitch dark at four o'clock in the afternoon and safe progress was not possible until eight o'clock in the morning, so the men had sixteen hours of weary waiting with not even the comforts of speech.

To add to their suffering a boat of provisions had escaped them and hunger preyed upon the victims. Five days of suffering in the cold, damp depths, with not a ray of sunshine to cheer them, these strong, hale men had become as mutes. It was not till now that the watchers above caught sight of the limping men who appeared about the size of jack rabbits. To call the attention of the men below one of the watchers threw down a small stone which loosed a larger one and this still a larger one, and so on until a ton of stone crashed into the water a hundred yards in front of the climbing men. A thrill of joy came to the lonely hearts below as they saw again the men in God's sunlight. After a waving of bandanas and a pistol salutation the explorers went on their way. It was now apparent that they could not proceed much farther and their energies must be bent on a means of escape if they hoped to save their lives.

Suddenly they came upon a large pile of rock which blocked the passage and under which the river coursed. For a moment all hope seemed abandoned. Mr. Torrence glanced at his men, and one glance was enough—despair was now stamped where courage and determination had so recently dwelt. After a few prayerful moments, on looking up, Mr. Torrence discovered what seemed to be an old water course leading precipitously into the canon twenty-five hundred feet deep. It was very narrow and in places inclined at an angle of eighty degrees. The course could not be traced to the top of the cliff, so they knew not whether it would terminate in a perpendicular cliff which could not be scaled, but with no other alternative of escape they ventured this one. Brevity forbids detail, but picture if you will a perpendicular wall 2,500 feet high with but a narrow precipitous path partially presenting itself to the climbers and, capping the climax, imagine night overtaking them as they clutched this wall two thousand feet above the roaring torrent, and five hundred feet beneath safety. Picture this, and you have an inkling of the plight of the climbing men. Long after dark, however, Mr. Torrence grasped the branch of a sage brush and with a shout pulled himself to safety, the others following, all drenched with perspiration.

They had covered but fourteen miles in twenty-one days. "This time the Black Canon won," declared Torrence, but he was determined yet to win.

Within a year after this perilous adventure, Mr. Torrence stood again in the upper canon to fight the battle over again. This time his only comrade was his fellow engineer, A. L. Fellows. They had especially designed for this trip a rubber air mattress provided with air-tight compartments for carrying provisions and also with hand straps which they could grasp to keep their heads above water.

For two weeks they fought against exhaustion and fatigue in the upper canon, when they reached the Falls of Sorrows, the point where the first party had abandoned the trip. Now a fearful unknown lay between them and their goal. The canon grew deeper and narrower and the increasing fall of the river was alarming as if it might be heading toward an underground waterfall. The two passed on, climbing, wading or swimming, and at times were immersed for hours in practically ice water. Had they not taken the precaution to lash themselves to the air mattress at times they would have sunk for exhaustion, never to have come up again.

Just as they had feared, anon the stream fell out of sight. They ventured as near the brink as they dared, but they could discern nothing but a roaring, swirling cataract. For the first time these engineers lost heart and sat with their faces buried in their hands. The only thing to do was to cast fortune on the waters and leap. Fellows leaped first, his body whirled in sight for a moment, and was gone. Torrence stood in awe, picturing to himself the mangled body of his friend. Unable to endure the suspense longer he loosed the mattress and followed his friend.

The men were whirled into a temporary unconsciousness, and upon recovering found themselves clinging to the rocks below the falls. For several hours they lay in sheer exhaustion through hunger and the fearful leap. For sixteen hours they had had nothing to eat and now they divided the last spoonful of cold beans between them.

After one more ordeal similar to, but more awful than the one just mentioned, the men climbed two thousand feet up the devil's slide at the foot of the canon, and this time the engineers had triumphed.

As a result of this heroic exploit, a tunnel over six miles long has been driven through the base of a mountain and the furious waters of the Gunnison are diverted through this tunnel into several hundred miles of canals and laterals to do the will of man. And on the 26th day of September, 1909, when President Taft touched an electric button the headgate of the tunnel was raised and Torrence and Fellows, standing with clasped hands at the Falls of Sorrows, just as they had done twelve years before, looked with immeasurable satisfaction upon the greatness of an accomplished work.

The Gunnison river project is one of the many projects now under course of construction by the Reclamation Service. While this project presented difficulties which are not to be met with in most other current undertakings, yet it exemplifies heroism and daring in superlative degree, which are indispensable to the successful irrigation engineer.

There are positively no "snaps" left on the "job." It requires capital, pluck, and brains to build irrigation systems nowadays.

Any of the failures, of which there have been too many in the last few years, can be traced to a lacking of one of these three things, and usually to the latter. Not infrequently when failure is apparently due to lack of capital, careful analysis will place the blame on the engineer. He has either not made sufficiently careful examination of his project to get substantial basis for his calculations and estimates, or else he is insincere and has willfully misrepresented conditions to the capitalists. In either case the result is disastrous. Construction work will be begun and when well under way, capital is being consumed faster than calculated, capitalists become discouraged and scheme so as to prevent heavy losses, the project goes into a receiver's hands, and irrigation finance has received another bump on the shin.

Had the estimates been accurate, or even exaggerated a little, the company taking the project would either have entered the work with sufficient funds or else turned it down flat, and irrigation bonds today would hold their place beside bonds of any other class, instead of being relegated to the rear and almost unmarketable.

I believe irrigation engineering offers equal or better opportunities for the energetic, daring young man than does any other branch of civil engineering. The field is yet new, competition low, and opportunities for individual enterprise are good..

DESIGN OF A STEEL STANDPIPE.

By Harry Schwerin, C. E. 1911.

1. A steel standpipe is a large tank, usually cylindrical in shape. generally used for the storage of water. The standpipe is used to good advantage in places where the elevation of the ground is not sufficient for the use of a reservoir. In such cases, the standpipe is made large enough to supply the town or city for the period of about one day, and sufficient to supply four or five fire streams. As the standpipes are easily made and erected in any place, they are made for a supply of not more than 10,000 or 12,000 population and duplicated if necessary.

2. The following data will be assumed in this design:

   1. Population of city 10,000.
   2. Domestic supply 60 gallons per capita per day.
   3. The supply to last 24 hours.
   4. The capacity must also be large enough for five fire streams with a discharge of 250 gallons per minute each.
   5. The water head must be sufficient to force a stream 70 feet high, which requires a pressure of 50 lbs. per sq. inch at the nozzle. For such a pressure, the head of water above the nozzle must be 116 feet.
   6. The diameter of pipe will be assumed as 24 feet.

3. **Height.** Having assumed the diameter, the proper height may be determined. Two things should be taken into consideration in computing the height. First, the capacity necessary for domestic use, second capacity necessary to maintain five fire streams for one hour. If we let:

n = Number of fire streams.

h' = Height in feet thru which water is drawn down by five streams in an hour.

d = Diameter of pipe.

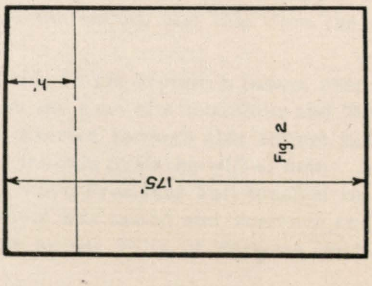


Fig. 2

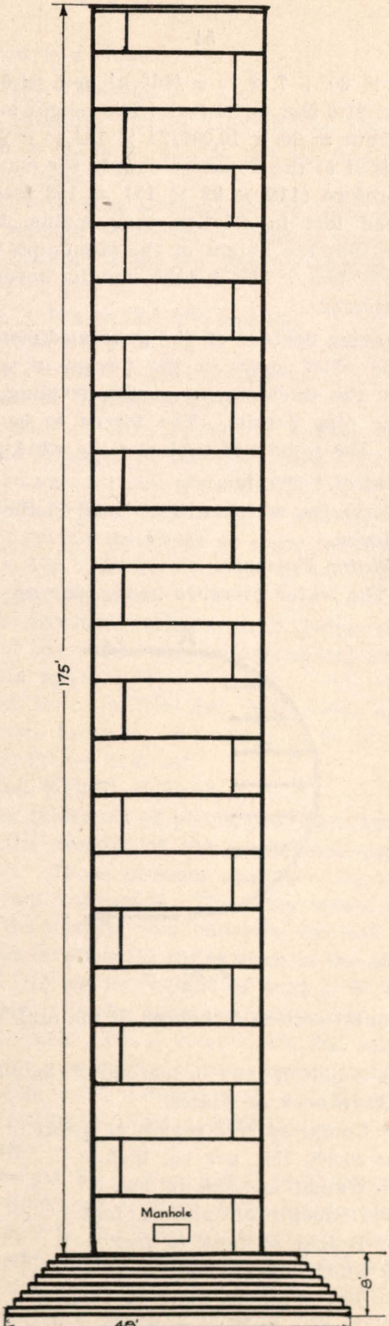


Fig. 1

Then $n \times 250 \times 60 \div 7.48 = \pi /4d^2 h^1$ or $n = 0.004d^2h'$; solving for h' we have $h' = 21.6$ feet or 22 feet. The height needed for domestic supply for one hour $= 60 \times 10{,}000/24 \times 144 \times \pi \times 7.48 = 7.4$ feet. 100% is usually added to the domestic supply for maximum use. The least height is therefore $(116 \times 22 \times 15) = 153$ feet. There is also some pressure head lost in friction thru mains, hose anl nozzle, and to account for that the height of the standpipe will be taken 22 feet greater, or 175 feet. The height should never be more than eight times the diameter.

4. **Design.** Having decided on the general dimensions the design may proceed. The chief steps in the design of a standpipe are: the calculation for the thickness of plates, riveting, the foundation, anchorage, and the pipe details. The forces to be considered are: the water pressure, the weight of tank and the wind pressure.

5. **Development of Formulae.**

Some of the formulae which will be used in the later discussion will now be developed.

Formula for Water Pressure.

If we let s = The water pressure in lbs. per sq. inch. Then at a

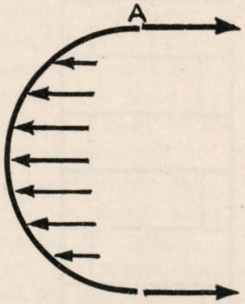


Fig. 3.

point A in a horizontal section in bottom of the tank $s = 62.5 \times h \times d/12 \times 2 = 2.6hd$.

Where h is the height of tank in feet and d the diameter in feet.

Formula for Thickness of Plates.

If we let Sc = Compressive strength of metal in lbs, per sq. inch.
= 20,000 lbs. per sq. inch.

If we let W = Weight of tank in lbs.

If we let d = Diameter of tank in feet.

If we let h = Height of tank in feet.

If we let e = Efficiency of joints = 70%.

If we let t = Thickness of plate in inches.

Then $t \times Sc \times d \times \pi = W$; $t = W/Sc \times d \times e \times 1/_{12\pi}$

$= 0.026 \times W/Sc \times d \times e$ ....(2)

Formula for Wind Pressure.

If we let $S_0 =$ Pressure due to wind in lbs. per circumferential inch, which is taken at 50 lbs. per sq. foot on one-half the projection of pipe.

If we let $M =$ Overturning moment due to wind in lbs. per foot.

If we yet $I =$ Moment of inertia of shell of pipe.

If we let $e =$ Distance from top fiber to neutral axis $= R$.

Then $So = Me/I$:

$M = So \times hd/_2 \times h/_2 = 12.5\ h^2d$ foot lbs.

$I = \pi R^3 = d^3/8$ and $e = R = d/2$.

$\therefore$ $So = 12.5h^2d8/_2 \times 12 \times \pi \times d^3 = 1.33h^2d$ ....(3)

6. **Thickness of Bottom Vertical Plate.**

With a safety factor of four, 15,000 lbs. per sq. inch may be allowed for the tensile strength of the steel used. The stress in lbs. per sq. inch in the bottom vertical plate will therefore be $S = 2.6hd/t \times e$ where 2.6hd is the water pressure (see art. 5), e the efficiency of rivet joints and t is the thickness of plate in inches. Then $t = 2.6hd/S \times e = 2.6 \times 175 \times 24/15{,}000 \times 0.7 = 1.04$ inches. A $1\ ^1/_{16}$-inch plate will therefore be used.

Since the water pressure varies with h, being zero when h is maximum, the rest of the plates may be determined graphically from the diagram shown in figure 4.

A–B being equal to h in feet and A–C equal to thickness of bottom in inches drawn to scale. All plates to be 96 inches from C. to C. of rivets, or 100 inches over all.

7. **Wind Stresses and Weight of Tank.**

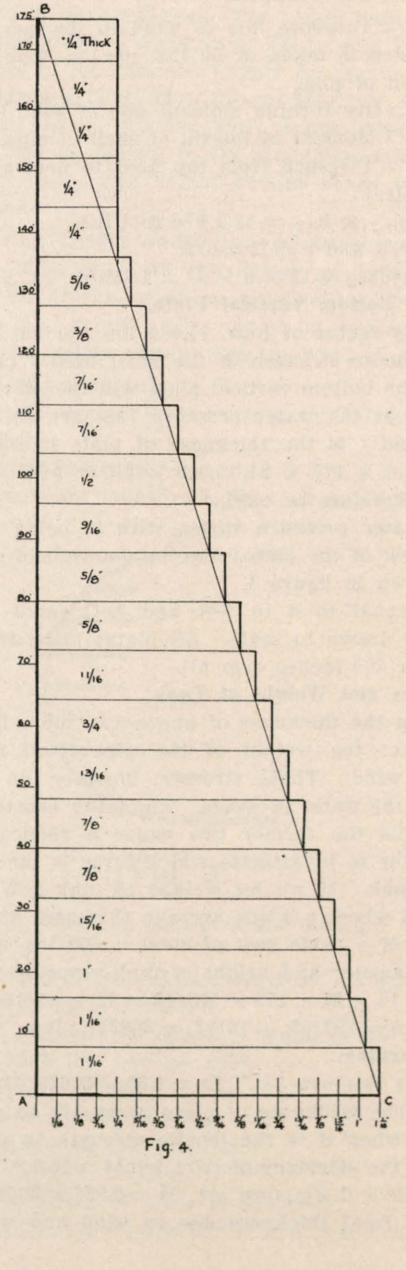
In computing the thickness of plates two more factors are to be taken in account: the weight of the pipe itself and overturning stresses due to wind. These stresses, however, do not add to the stress caused by the water pressure. The latter exerts a tensile stress horizontally, while the former two cause a vertical stress. It is, therefore, a matter to investigate which force is the greater.

8. **Weight of Tank.** If we let weight of tank $= W$ then $W = \pi \times t \times d \times h \times V/12$ where t is the average thickness of plate in inches. V is the weight of a cubic foot of steel = 500 lbs. approximately, d and h are the diameter and height of tank respectively, both in feet.

$\therefore$ $W = 3.1416 \times 17 \times 24 \times 175 \times 500/12 \times 32 + 3.1416 \times 24 \times 41 \times 500/12 \times 8 = 292108. + 16103. = 308211.$ lbs.

Wind Overturning.

$So = 1.33h^2/d$ (see art. 5) $\therefore$ $So = 1.33 \times 30625/24$ lbs. per circumferential inch. The thickness of plate necessary to carry this stress is $t = So/S \times e$ when S is the tensile strength in metal in lbs per sq. inch and e is the efficiency of rivet joints = 15,000 lbs. per sq. inch, $\therefore t = 0.026W \div Sc \times d \times e$ (see art. 5) $= 0.026 \times 306211/20000 \times 24 \times 0.7 = 0.02$ inch. Total thickness due to wind and weight of tank =



175'
170'
160'
150'
140'
130'
120'
110'
100'
90'
80'
70'
60'
50'
40'
30'
20'
10'
B
A
C
1/4" Thick
1/4"
1/4"
1/4"
1/4"
5/16"
3/8"
7/16"
7/16"
1/2"
9/16
5/8"
5/8"
11/16"
3/4"
13/16"
7/8"
7/8"
15/16"
1"
1 1/16"
1 1/16"

Fig. 4.

$0.02 + 0.16 = 0.18$ inch, which is less than the thickness necessary to resist the water pressure, and, therefore, will be neglected.

10. **Riveting.**

Note:—All plates from ¼″ to $\frac{5}{16}$″ thick use ⅝ rivet; all plates from ⅜″ to $\frac{7}{16}$″ thick use ⅜ rivet; all plates from ½″ thick and above use ⅞″ rivet.

Rivets should not be spaced over 3″ apart, C to C.

Rivet holes should be drilled in plates above ¾″ thick.

11. **Horizontal Riveting of Bottom Plates.**

Since the horizontal joints are not stressed by the water pressure, lapped joints will be sufficient, as shown in figure 5. Stresses to be considered at this joint are those due to wind overturning and those due to weight of tank.

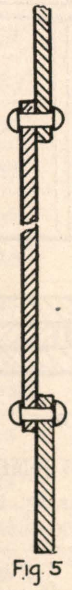


Fig. 5

If we let S' = total stress on horizontal joint $S' = (So \times Sw)$ where Sw is the stress per circumferential inch due to weight or tank. $So = 1.33h^2/D = 1{,}700$ lbs. per circumferential inch, $Sw = W/12 \times \pi \times d = 308211/12 \times \pi \times 24 = 341$ lbs. per circumferential inch, therefore, $S' = 2040$ lbs. per circumferential inch.

12. **Spacing of Rivets on Horizontal Joints.**

If A be the distance between centers of rivets, then $A = 6013/2040$ where 6013 is the value of a ⅞″ rivet in single shear; therefore, $A = 2.94$ inches from C to C of rivets. The spacing will be the same on all horizontal joints.

13. **Vertical Riveting.**

The vertical joints must resist the water pressure. A lap joint will not be sufficient, and a double riveted butt joint should be used as shown in figure 6.

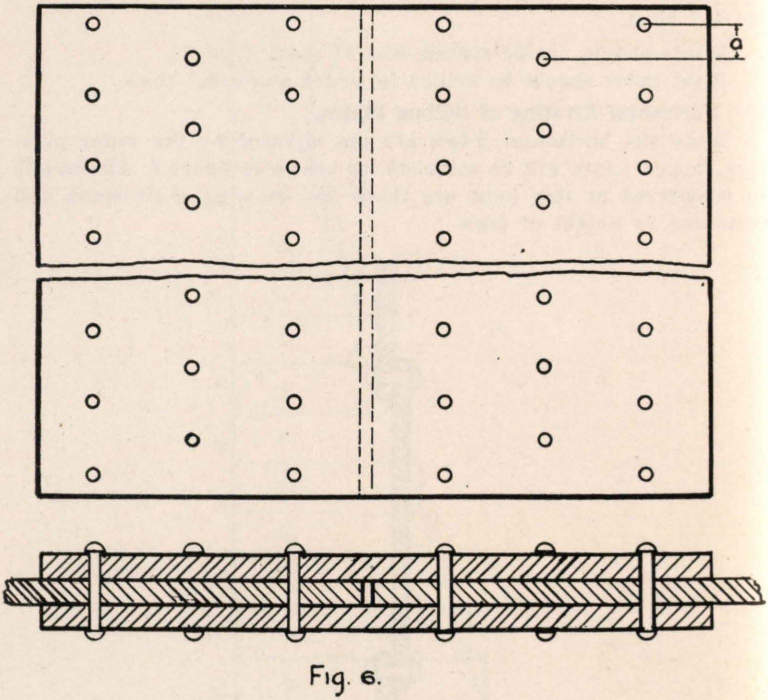


Fig. 6.

14. **Spacing of Rivets on Vertical Joints.**

If Ss = Unit shear in lbs per sq. inch = 10000 lbs.

If St = Tensile strength 16000 lbs. per sq. inch.

If Dr= Diameter of rivet ⅞″.

If T = Thickness of plate 1 $\frac{1}{16}$″.

If E = Efficiency of Joint.

If N =Number of rows.

If a = Spacing as shown in figure 6.

Then $T \times (a-d) \times St = N \times \pi/4 \times d^2 \times 2Ss$. (2Ss is taken in this equation because the rivets are double shear.) $\therefore A = n \times \pi \times d^2/4$ $\times t \times 2Ss/St \times d = 3 \times 3.1416 \times 49 \times 16/64$ $\times 17 \times 4 \times 2 \times 10000/16000 \times 7/8 = 2.905$ inches from C to C of rivets.

The spacing of the other plates may be computed in the same manner using the corresponding values of Dr, t and N. The value of

N may be found by trial only. Some tables may be obtained, as those issued by "The Chicago Bridge and Iron Works", in which spacing of rivets, number of rows, and size of strap plates are given. Their results check closely to the results obtained from above formula.

TABLE SHOWING NUMBER OF RIVETS, SPACING OF RIVETS AND STRAP PLATES.

Plate No.	Thickness of Plates	No. of Rows	Spacing	Dimensions of Strap Plates
1	1 1/16"	3	2.905"	16.5" x 5/8"
2	1 1/16"	3	2.905"	16.5" x 5/8"
3	1"	3	3.10"	16.5" x 9/16"
4	15/16"	3	3.27"	16.5" x 9/16"
5	14/16"	3	3.37"	16.5" x 1/2"
6	14/16"	3	3.37"	16.5" x 1/2"
7	13/16"	2	3.65"	11.5" x 7/16"
8	3/4"	2	2.86"	11.5" x 7/16"
9	11/16"	2	2.05"	11.5" x 7/16"
10	10/16"	2	2.05"	11.5" x 7/16"
11	10/16"	2	2.05"	11.5" x 7/16"
12	9/16"	2	2.05"	11.5" x 7/16"
13	8/16"	2	2.05"	11.5" x 7/16"
14	7/16"	2	2.05"	11.5" x 7/16"
15	7/16"	2	2.05"	11.5" x 7/16"
16	6/16"	2	2.05"	11.5" x 7/16"
17	5/16"	2	2.05"	11.5" x 7/16"
18	1/4"	2	2.05"	11.5" x 7/16"
19	1/4"	2	2.05"	11.5" x 7/16"
20	1/4"	2	2.05"	11.5" x 7/16"
21	1/4"	2	2.05"	11.5" x 7/16"
22	1/4"	2	2.05"	11.5" x 7/16"

15. **Base.**

The base is attached to the sides by two angles to better distribute the dead load weight over the masonry foundation. The angles used are 5″ × 5″ × ⅝″ as shown in figure 7.

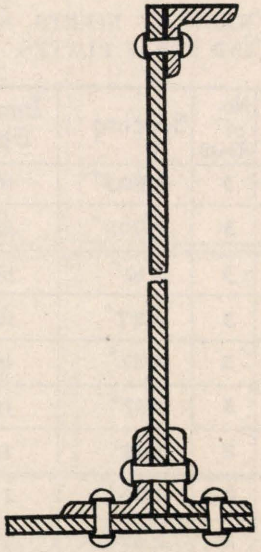


Fig. 7

The base is usually made of ½″ plate to which the inlet and outlet pipes are attached in the manner as shown in figure 8.

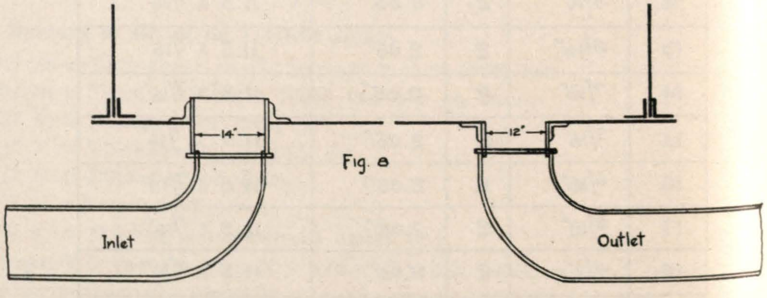


Fig. 8

16. **Top.**

To prevent the top from collapsing from the force of the wind, an angle usually a 4x4x⅜ is riveted to the outside.

17. **Anchorage.**

In tanks of large diameters the anchorage may be neglected, but where the diameter is small compared with its height, the overturning moment of the wind may be greater than the weight of the tank when empty. It is therefore necessary to fasten it to the foundation. A good arrangement is the bracket shown in figure 9.

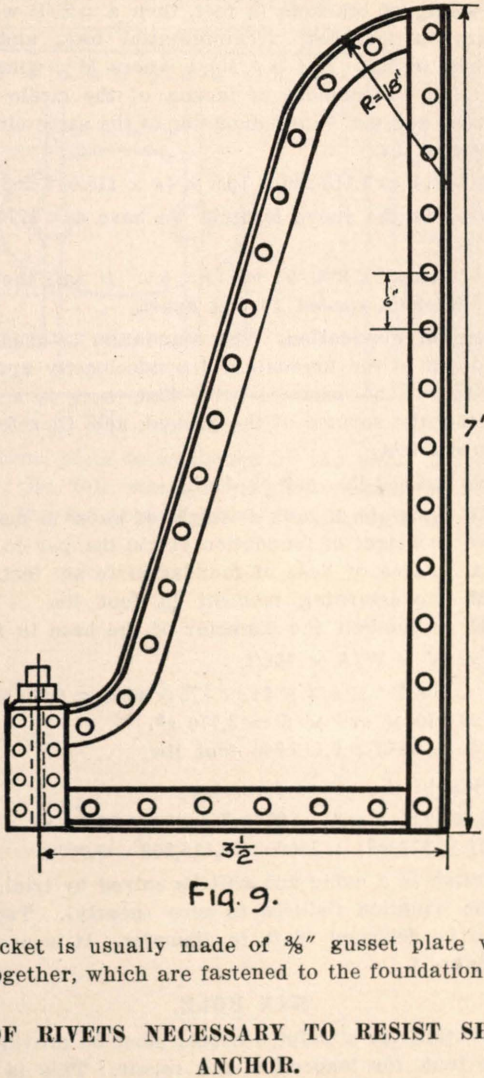


Fig. 9.

The bracket is usually made of ⅜" gusset plate with 4"x3"x⅜ Ls riveted together, which are fastened to the foundation by 2" anchor bolts.

NUMBER OF RIVETS NECESSARY TO RESIST SHEARING IN ANCHOR.

If T = tensile strength of anchor bolt in lbs. per sq. inch. Then $T = 15{,}000 \times \pi v^2 = 47{,}200$ lbs. and the rivets necessary to carry this stress $= 47{,}200/6013 = 8$ rivets. To insure safety, however, it is good practice to make the brackets 7 ft. high with a base of $3\frac{1}{2}$ ft., rivet spacing not to be less than 6 in. C to C.

SPACING OF BRACKETS ON A 31 FOOT CIRCLE.

If $a =$ spacing of brackets in feet, then $a = T/P$ where P is the wind pressure in lbs. per circumferential foot, and $T =$ tensile strength of bolt in lbs. But $p = Me/I$ where $M =$ wind overturning moment on tank, $I =$ moment of inertia of the circle passing thru the anchor bolts, and $e = \frac{1}{2}$ the diameter of the same circle. $M = 12.5$ hd $= 9{,}115{,}260$ foot lbs.

$I = \pi v^4 \div 4 \therefore P = 9{,}115{,}260 \times 15.5 \times 4\pi \times (15.5)^4 = 3141.3.$

Substituting in the above formula we have $a = 47200/3141.3 = 15$ feet.

Number of brackets will be $\pi d/15 = 6.5$. It will therefore, be designed for 7 brackets spaced 14 feet apart.

19. **Design of Foundation.** The foundation is usually made circular. The depth of the foundation depends mostly upon the nature of the soil. It will be assumed here, that there is a good bearing soil 8 feet below the surface of the ground, and, therefore, the depth will be assumed 8 feet.

Now, if $Sb = 5{,}000$ lbs. per sq. foot.

" W = weight of tank + weight of water in lbs.

" w = weight of foundation at 150 lbs. per cu. ft.

" A = area of base of foundation in sq. feet.

" M = overturning moment in foot lbs.

" E = one-half the diameter of the base in feet.

Then $Sb = W \times W/A \times Me/I.$

$W. = 308{,}211 \times \pi/4 \times 24, \times 175 \times 625) = 5{,}256{,}231$ lbs.

$w = 150 \times \pi r^2 \times 8 = 3{,}770\ r^2.$

$M. = 12.5h^2d = 9{,}115{,}260$ foot lbs.

Substituting in the above formula:

$5000 = (5{,}265{,}231 \div \pi r^2) \times 3770r^2 + (9{,}115{,}260 \times r) \div \pi/4 \times r^4$ and $r^2 = (5{,}256{,}231 \times 3770r^2) \div 5000\pi + 9{,}115{,}260 \div 5{,}000r.$

This equation is a cubic and will be solved by trial. By assuming $r = 24$ the equation reduces to zero (nearly). The foundation, therefore, will be designed 48 ft. in diameter. It is usually stepped as shown in figure 1

MAN HOLE.

In constructing the standpipe access must be provided for to the inside of the tank for inspection and repair. This is obtained by making a hole in the bottom vertical plate large enough for the passage of a man. According to its function it is called man hole. It is good practice to make it elliptical in shape. The hole is covered by an overlappng lid, which is held in place by the inside pressure, but to make it water tight it is fastened with one or two bolts to an outside curved beam as shown in figure 10.

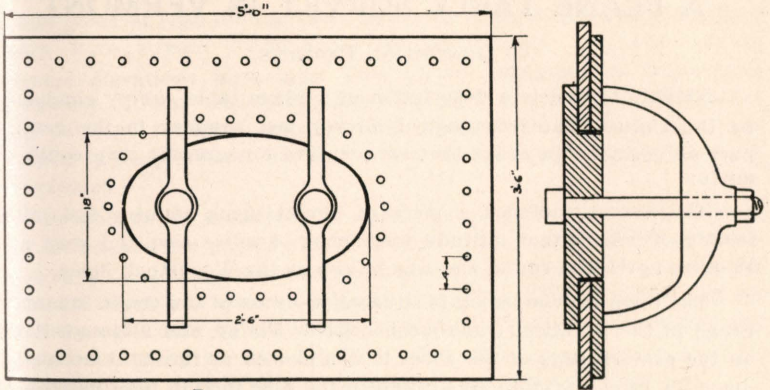


Fig. 10.

The bottom plate is weakened by the metal taken out from it. To develop the full strength a plate should be riveted around the opening, the net area of which should equal the area taken out.

A PLANE TABLE SURVEY IN VERMONT.

JOSEPH M. PERKINS.

THE following is a description of a plane table survey conducted by the United States Geological Survey last summer in the central part of Vermont, in order that an accurate topographic map could be made:

The survey included a piece of mountainous country five miles square, which in that latitude was about 14 miles east and west and 18 miles north and south, and was known as the Woodstock Sheet.

The town of Woodstock is situated just east of the Green Mountain range in the beautiful Ottauquechee River Valley, and although it lay on the eastern edge of the sheet it was chosen as the most accessible place for headquarters. At first all of the nine men in the topographic party lived in Woodstock and surveyed the nearby country, but later they moved to other places more convenient to their work, staying sometimes in small towns and often in farm houses, but they usually drove back to Woodstock each Saturday night to consult with the chief and have their work assigned to them.

The survey was started in the early part of July from one government benchmark and three triangulation points. The bench was just over the east edge of the sheet and was established several years ago by another party. The three triangulation points were also at short distances from the sheet. From these three points other points in the sheet were located by means of a plane table with a large board and an alidade with a telescope. To accomplish this, large white flags were placed on some of the high hills. These were intersected from other peaks and plotted on the large plane table sheet by the chief, who was usually accompanied by a rodman. Other prominent landmarks, such as church spires, isolated buildings, and lone trees were located in the same way. Meanwhile other parties had started the primary levels and the road traverse.

The primary levels were started at the known bench, and carried the elevations to each quarter of the sheet, making several closed circuits, and establishing primary bench marks. There was at least one primary bench established in each six-mile square. These bench marks were made of aluminum, and in shape resembled a mushroom. After drilling a hole in a rock or large boulder, the bench mark was set in place, stem down, and cemented firmly with neat Portland cement.

There were two men in the level party, a levelman and a rodman. The levelman used a twenty-two inch wye level and both he and the rodman read the rod and kept notes. The greatest error of closure allowed on primary levels was 1-500 distance in miles. Turning points were taken on an iron pin driven into the ground, and computed elevations were painted in white along the road on rocks, trees and telegraph poles, at road intersections, summits, depressions, and also where there were stream crossings or abrupt changes of grade. These painted elevations were later used by the topographers while road sketching. It was usually necessary for the level party to have a horse and carriage to carry them to and from work, and the horse was trained to keep a short distance ahead of the level party while work progressed.

Secondary or flying levels were run over all roads which were not included in the primary level circuits. In secondary leveling an 18-inch wye level was used, and the rodman carried a 20-foot extension rod. No effort was made to balance foresights and backsights, and the rod was read only to hundredths of a foot. This was fast work and extreme accuracy was not required. Elevations were painted in white along the road, just as was done by the primary level party.

In making a topographic map of this kind it is essential that all roads shall be surveyed and plotted. This was done by two traversemen, working independently and alone, each with a small plane table and a horse and carriage. This traverse was begun at a road intersection which had been previously located as the starting point on the plane table sheet. The plane table was oriented with a restricted compass which was attached to the board. The board was leveled by shifting the tripod legs until the compass needle swung freely. The needle indicated magnetic north, but true north was plotted on the plane table sheet.

A distant point in the road was sighted with the alidade and a line was drawn in that direction, its definite length being determined by counting the revolutions of the carriage wheel, and plotting them to scale with the aid of a conversion table. These conversion tables are published by the survey in pamphlet form and after finding the circumference of the wheel used, the corresponding table may be cut and fastened to the board to facilitate the plotting. The traverseman would not usually set up at the point sighted ahead, but would measure the distance to it and set up at a point beyond, and sight back to it. Thus a setup was made only at alternate stations, the other stations being platted by intersection. This saved considerable time and was as accurate as was consistent with the scale used, which was 1-4800.

Each time a setup was made, the road was first plotted, then side shots were taken when necessary to intersect other points such as hill tops, buildings remote from the road, the corners of woodlots, or flags which were already located on the large plane table sheet. It was the custom to tie in hilltops by three intersecting lines drawn from the different setups and intersecting with wide angles. In this manner a road traverse of the entire sheet was made.

After the road traverse was adjusted and transferred to another sheet, the chief started the contour sketching. Much of this sketching could be done from the roads, using the painted elevations left by the level parties. Occasionally the height of a hill would have to be determined with aneroid barometer. When this was done, the barometer was first set to agree with a painted elevation at a point in the road, and was then carried to the top of the hill and read. Road sketching also included the locating of streams and lakes, and any other important details that could be seen from the road.

High ridges which were inaccessible from the road were traversed by a topographer and a rodman. The rodman went ahead with the forward end of a waxed linen tape 528 feet long. The topographer with the plane table and aneroid barometer, made a traverse of the top of the ridge, locating contours as he went along. Most of these ridges were thickly wooded, and a setup had to be made every time the rodman went ahead with the tape. The ridge traverse was connected at each end with the road traverse.

Thus the work continued throughout the summer, until one day early in October word came from Washington to stop work for the season.

INVESTIGATION OF THE STRENGTH OF NAILED JOINTS.

* BY EMIL OEFFINGER, C. E. '11.

AMONG the problems of original investigation in our course of testing materials, is the investigation of the strength of nailed joints, which is thought to be interesting to the reader. In spite of the fact of the wide application of nailed joints, there is at present a very limited amount of data in text books, pertaining to their strength. The reason for this is not evident, unless it be that the great variety of conditions involved makes it impracticable. Some of these variables are: kind, size, and condition of wood, kind, size and spacing of nails, clinching, workmanship, etc.

In order to ascertain the effect of various conditions on the strength of nailed joints, it would require a comprehensive series of tests to be performed. From a small number of tests it is impossible to depend on their results, as a number of influential factors enter into the determination and cannot be eliminated although the greatest precision be exercised. The reliability of the results derived from a small number of tests, would depend to a considerable extent upon the selection of material and the personal equation of the one conducting the tests.

The tendency in practice is to use an excess amount of nails, presumably for safety's sake. The quantity introduced in any joint is a function of the judgment of the carpenter or workman. He uses such a number of nails, as he knows from actual experience will be capable of carrying the required stress. The objection to this method is its unreliability and lack of economy. Not enough consideration is given to the size of joints. For instance two nailed joints are required to hold the same amount of stress, and the one joint may have a larger contact area than the other; the carpenter or workman will introduce nails in the larger joint in proportion to the area of contact with the smaller joint, while theoretically the additional quantity of nails used in the larger joint, should be proportional to the difference of weight of the materials in the two joints.

In another instance however, an excess of nails is used because of the inability of the workman to determine the maximum load the

---

*Problems for original investigation in Laboratory for testing materials, assigned to H. Muller, S. B. Ehrenrich and Emil Oeffinger, Civil Engineering, '11.

nailed joint is required to carry; but again the difficulty in calculating the stress may render it cheaper to use an excess.

If access were had to some simple, reliable data concerning the strength of nailed joints, the number of nails necessary could be computed for that particular joint, thereby reducing this loose method of approximation to a rational method. The advantages would appear in saving of materials and labor, which are two important factors in any structure.

The object of these tests was merely to gain a preliminary insight on the properties of nailed joints, as the number of tests were too limited to deduce any authoritative relations. The different conditions that were investigated on the strength of nailed joints were as follows:

The effect of different kinds of wood; the effect of number of nails; the effect of size of nails; the effect of clinching nails; the effect of spacing nails; the effect of different kinds of joints; the effect of moisture.

In order to obtain comparisons, six of these conditions were kept constant and the seventh varied.

Test pieces consisted of selected lumber, and thirty-four tests were made. Three kinds of woods were used: Red Oak, Yellow Pine, Basswood. All specimens were free from windshakes, knots and seasoning checks. The number of rings per radial inch were: yellow pine 16; basswood 12; red oak 14; approximately 12% of moisture.

In studying the effect of the number of nails in the strength of nailed joints, 4, 6, 8, 10, 12, nails were used. They were spaced as follows:

Four Nail Spacing:—Nails started one inch from the end, and two inches from sides, two nails in the first row. Second row of nails symmetrical to the first. Distance from nail line to nail line two inches.

Six Nail Spacing:—Nails started one inch from the end, and one inch from the sides. The rows of nails were staggered. Three nail lines contained six nails forming an isosceles triangle. Distance from center to center of nail lines was two inches.

Eight Nail Spacing:—First row started one inch from the end and one inch from the sides, 4 nails in a row and two rows containing 8 nails, symmetrically placed with number one. Distance between nail lines was two inches.

Ten Nail Spacing:—First row started one inch from end and one inch from the sides. Nail rows were staggered, four nail lines containing ten nails, forming an isosceles triangle. Distance between nail lines was one inch.

Twelve Nail Spacing:—First row of nails started one inch from the end and one-half of an inch from the sides. Three rows contained twelve nails. Nails were staggered forming a trapezoid. Distance from nail line to nail line was one inch.

In all cases the nails located on the nail lines parallel to the end of the board were equidistant from each other.

Test pieces consisted of two boards 12"x6"x1". Nails were driven perpendicular to the plane of the board, thereby eliminating any error that might be introduced through the non-uniformity of the direction of nails. Two types of nailed joints were used

(1) Grain parallel, called Lap joints.

(2) Grain perpendicular, called T joints.

(See Drawing, Plate I.)

The parallel grained joints were allowed a lap of four inches, perpendicular grained joints one and one-half inches, the size of the boards remaining constant. It will be evident that the areas of contact are not the same, rendering the friction between the boards different in each type of joints.

The effect of the size of nails or the strength and dimensions of these nails are tabulated in table Number Two. The nails were of the common wire type.

The method of testing the nail joints was restricted to shear. All specimens were eccentrically loaded (see Plate 4). Spherical head was used throughout the test.

An initial load of 100 pounds was applied. Deformations were measured to thousandths part of an inch, Riehle compressometer. The attachment of the compressometer was effected by nailing a small strip of wood to the lower end of the board forming the upper part of the joint (see Plate 4).

Increments of one hundred pounds were applied slowly, by hand power, until the load ceased to be proportional to the deformation, then the compressometer was removed and the joint tested for its maximum load with a higher speed and power.

All tests were made with a Riehle Vertical Screw Testing machine, in the engineering laboratory of Valparaiso University. Load deformation curves were plotted for each test and numbered to correspond to specimens. (See Plates 2 and 3).

Specimens 1, 2, 3 were constructed of yellow pine, basswood and red oak. 8—8d nails were used, nails unclinched. Lap joints were used.

Specimens 4, 5, 6:—The conditions involved in these joints were the same as in the preceeding ones, with the exception that a T joint was substituted in place of a Lap joint. From these conditions we can obtain a comparison between the strength of Lap joints and T joints.

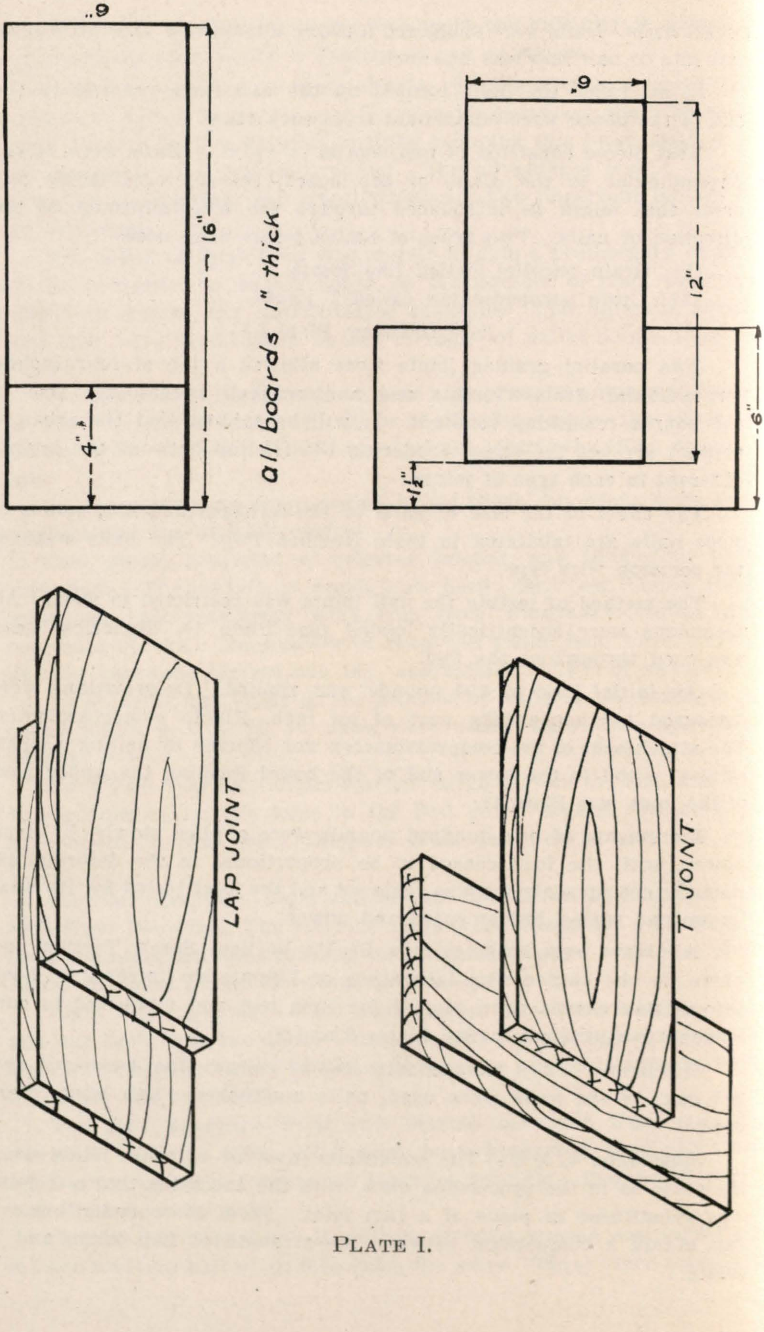


PLATE I.

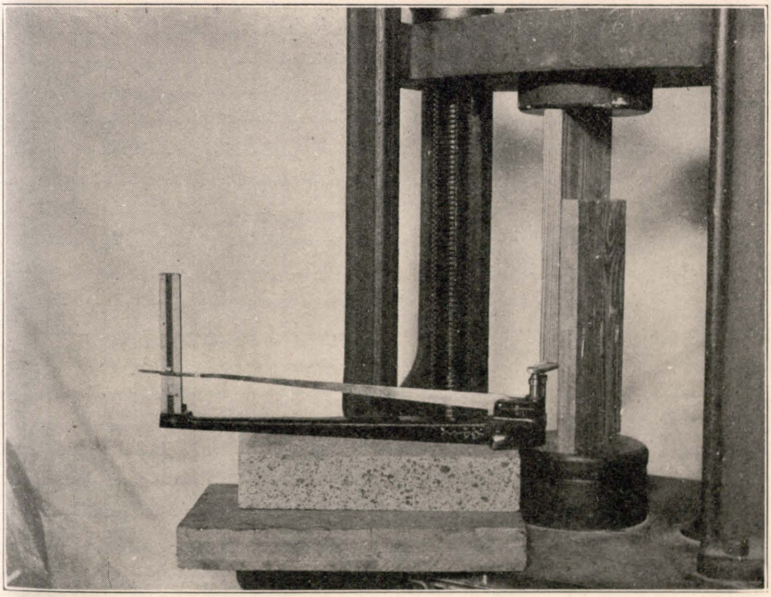


Plate IV.

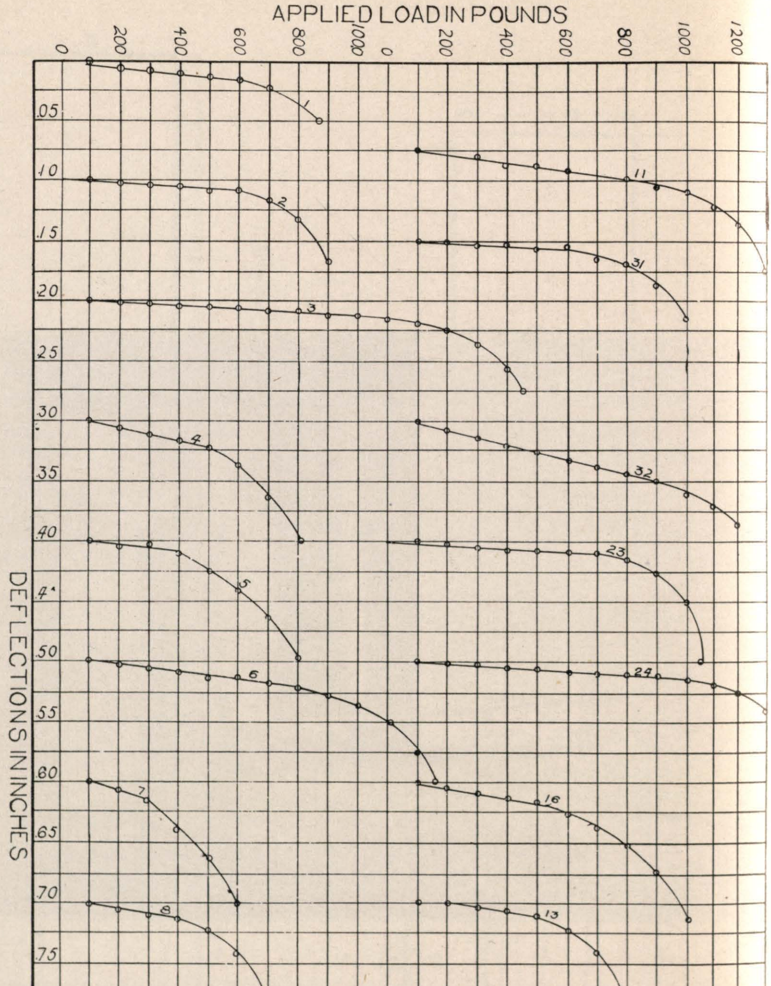
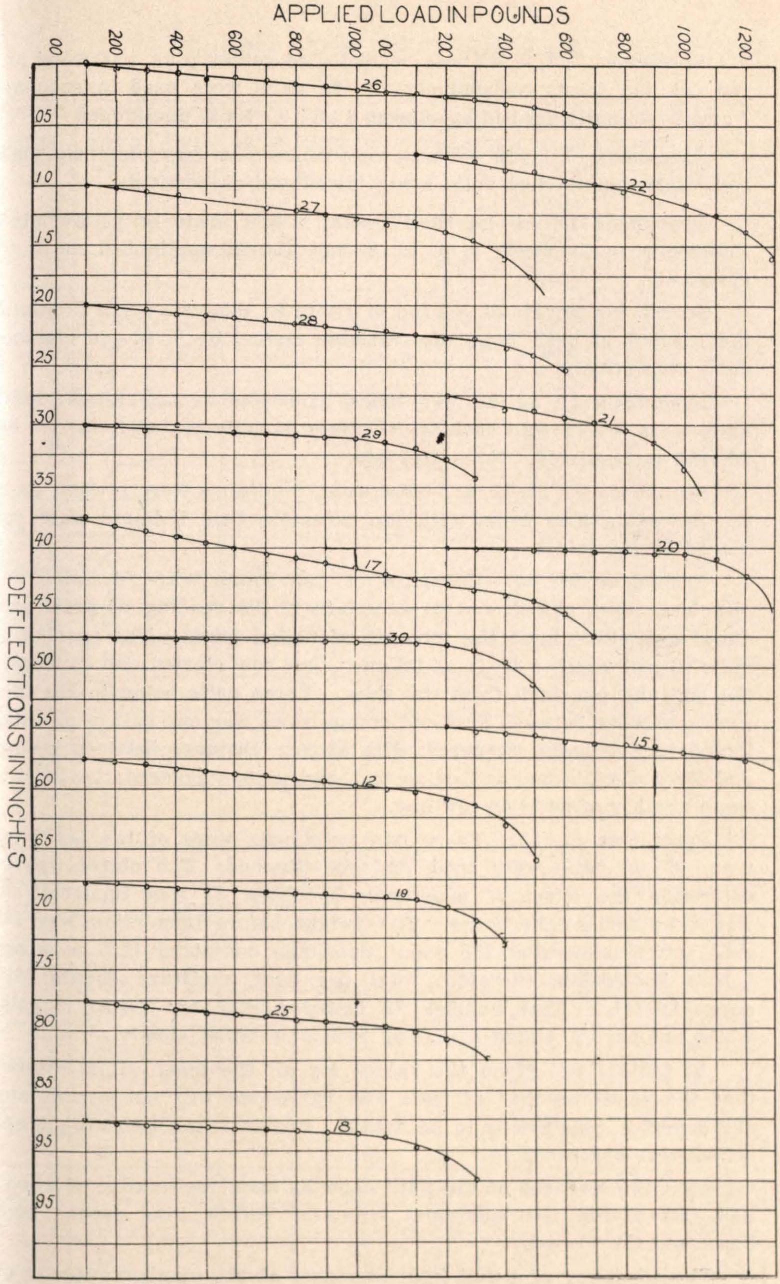

APPLIED LOAD IN POUNDS
0
200
400
600
800
1000
0
200
400
600
800
1000
1200
DEFLECTIONS IN INCHES
.05
.10
.15
.20
.25
.30
.35
.40
.50
.55
.60
.65
.70
.75
.85
.90
.95
100
1
2
3
4
5
6
7
8
9
10
11
31
32
23
24
16
13
14

PLATE II.



APPLIED LOAD IN POUNDS
00
200
400
600
800
1000
00
200
400
600
800
1000
1200
DEFLECTIONS IN INCHES
05
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
95
95
26
22
27
28
21
29
20
17
30
15
12
19
25
18

PLATE III.

Specimens 7, 8, 9:—These consisted of yellow pine, basswood and red oak lap joints respectively. 4—8d nails were used in specimen 7 and 6—8d nails used in specimens 8 and 9. Nails unclinched.

Specimens 10, 11, 12:—These were yellow pine Lap joints, six, eight and twelve unclinched nails being introduced respectively.

Specimens 13, 14, 15, 16, 17:—Lap joints made of yellow pine. Number of nails used: 4, 6, 10, 8, and 12—8d unclinched nails respectively.

Specimens 18, 19, 20:—Type of joints in this case were T joints; the number of nails being the variable factor, 10, 8, 12—8d clinched nails respectively.

Specimens 21, 22, 23, 24:—These consisted of Lap joints; eight nails were driven into each joint. Sizes of nails are as follows: 6d, 8d, 12d respectively. Nails clinched.

Specimens 25, 26, 27, 28:—The same conditions were present as in the preceding four tests, with the exception that T joints were replaced by lap joints.

Specimens 29, 30:—Consisted of Lap joints with 8d nails, unclinched. The object was to ascertain if the spacing of nails produced any effect upon the strength of nailed joints. The spacing of nails in test number 29, is as follows: first row started one inch from the end and one inch from the sides. Three nails being in the first row. Distance between first and second rows was one-half of an inch, including two nails staggered, with above. Distance between second and third rows is one-half of an inch and nails staggered; having the same number of nails as row one.

Specimens 31, 32:—These two specimens were of the lap joint type. 8—8d nails were used, and not clinched. The object was to determine the effect of moisture. Specimen 31 was immersed in water for forty-eight hours. The weight before immersion was 589 gms., after immersion 732 gms. Specimen contained 12% moisture before immersion, therefore total per cent. moisture equals 36% approximately. Test number 32 tested higher but failed rapidly, while number 31 tested lower by 24% and failed slowly.

In table I are given the values for all the tests. It is evident that the small number of tests and variations will not permit any authoritative conclusions to be drawn. However, the following properties were observed:

That the strength of the joint depends upon the quality of wood, Red Oak giving the maximum strength, Yellow Pine second, and Basswood the lowest.

The efficiency of a nail joint increases as the number and size of the nails increase.

SUMMARY OF STRESSES

NO. OF TEST.	NO. OF NAILS	SIZE OF NAILS	LAP JOINT	TEE JOINT	NAILS CLINCH'	NAILS NOT CLIN'	KIND OF WOOD	NAIL SPACING	MAX. LOAD
1	5	8	"			"	YELLOW PINE		1180
2	5	"	"			"	BASS WOOD		1120
3	5	"	"			"	RED OAK		2370
4	5	"		"		"	YELLOW PINE		1090
5	5	"		"		"	BASS WOOD		920
6	5	"		"		"	RED OAK		1945
7	4	"	"			"	YELLOW PINE	Spacing constant except Tests numbers 29.30.31.32	910
8	4	"	"			"	"		700
9	6	"	"			"	"		1200
10	6	"	"			"	"		1960
11	8	"	"			"	"		2250
12	12	"	"			"	"		3000
13	4	"	"		"		"		1100
14	6	"	"		"		"		1650
15	10	"	"		"		"		3140
16	8	"	"		"		"		2500
17	12	"	"		"		"		3750
18	8	"		"	"		"		2500
19	12	"		"	"		"		3420
20	10	"		"	"		"		3080
21	8	6	"		"		"		1650
22	8	8	"			"	"		2080
23	8	12	"			"	"		2300
24	8	16	"			"	"		2450
25	8	6		"		"	"		2440
26	8	8		"		"	"		3250
27	8	10		"		"	"		2610
28	8	12		"		"	"		3400
29	8	8	"			"	"		2400
30	8	"	"			"	"		2700
31	8	"	"			"	"		1640
32	8	"	"			"	"		2100

TABLE NO. II.

Size of Common Wire Nails.

SIZE	LENGTH	*GAUGE	NO. TO ONE LB. (APPROX.)
2	1 inches	0.072	876
3	1¼ "	0.080	568
4	1½ "	0.098	316
5	1¾ "	0.098	271
6	2 "	0.113	181
7	2¼ "	0.113	161
8	2½ "	0.124	106
9	2¾ "	0.124	96
10	3 "	0.148	69
12	3¼ "	0.148	63
16	3½ "	0.162	49

*NOTE:—Dimensions of sizes in decimal parts of an inch.

The clinching of nails increases the ultimate strength of a joint only, possibly 20%.

The strength of a nail joint varies with the spacing of nails: wider spacing being stronger.

Lap joints (parallel grained) are more efficient that T joints (perpendicular grained).

Moisture lowers the strength of the joint; this is undoubtedly due to the fact that the bearing power of the wood is weakened.

YOUNG ENGINEERS AND THE PHILIPPINES.

BY E. C. EARLE, C. E. '11.

The problem confronting the average engineering graduate is where to start; where to start that he may gain experience and at the same time put to advantage his four years of college training. While the principle of start at the bottom and climb the ladder is right and applicable, yet in consideration of the training received, it seems to our graduate hardly right that he should start at driving stakes, or some similar position with a salary less than that of the ordinary layman. There are some who by good fortune or influence, obtain very good positions at once, and make good, but the average student graduate usually has to start at the bottom. A good foundation is a safety factor in the future development of the young engineer and it is best that he start in the lower positions, for if he has the ability, promotions will follow, as he demonstrates the same to his superiors. But many times these promotions come slow, which is discouraging and irksome. Eventually the college training puts the man ahead, but primarily it counts for little. It is safe to say that in many cases our young men have been held in subordinate positions for an unreasonable length of time on account of their college obligations. The young engineer needs experience. Valuable experience is obtained by observation of engineering projects and methods. To secure this, travel, which is not always within easy reach of the young graduate, is essential.

The Philippines are calling for young American engineers, offering them an excellent opportunity; affording travel and experience, and giving them in return for their services a salary sufficient to get a fair start in the world, enough so that they may repay their college obligations without being tied down to drudgery for several years. Not only does the Philippine Civil Service Commission offer an excellent salary, affording travel, etc., but such working hours as to leave leisure time for study and self-improvement for future advancement and success. This year the Commission is calling for Civil Engineers and Surveyors and making appointments at $1,400 entrance salary. The Government pays for transportation of their engineers to Manila, in addition to their salary.

The only condition placed on the appointed engineers is to give two years' satisfactory service in the Islands. Most of these positions are

---

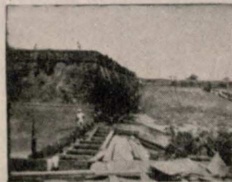
Note. Photos herewith reproduced are the joint production of E. C. Earle, and G. W. Vilian, a former Manila associate and member staff Improvement of Port Works.

filled by graduating engineers, and as a rule an increase to $1,600 may be expected after six to twelve months' service. A year to a year and a half and at the most two years, should bring a further increase to $1,800. After that, it is up to the individual to make good for future advancement. In addition to this excellent offer, there is a sick leave allowed on full pay and a vacation of thirty to thirty-five days, which accrues. If, at the end of three years' service, this vacation has not been used, a vacation of three months on full pay may be taken in the United States and an additional two months will be allowed on half pay for time consumed in traveling to the United States and back to the Islands.

Think of it! Are these not opportunities?

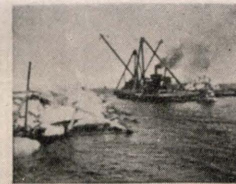
Why is it that the Philippine Civil Service Commission has experienced so much difficulty in securing enough engineers for the Islands? There is but one answer, ignorance among our young men of the prevailing conditions, and exaggerated ideas that the Philippines are unhealthy and a network of swamps. This I have found to be the general idea among the majority of Americans. In other words, there is a feeling of dread for the Islands, which prevents our men from grasping the great opportunities offered. That these ideas are erroneous is needless to say and after five years' experience in the Philippines, I may say with assurance that the conditions are grossly misrepresented and exaggerated in America. If I may correct these false impressions concerning the condition of the Islands, thereby bringing into their reach a larger field of opportunity, I will consider this short article as having accomplished its purpose, for I cannot attempt herein a description of facts other than those of vital importance to young engineers. The trip is a most delightful and beneficial one. Twenty-five or thirty days are required to make the journey from either Seattle or San Francisco by the large and commodious steamers of the Pacific (see photo S. S. Dakota map and route), with stops at Honolulu, Hawaiian Islands, Yokohama, Kobe, Nagasaki, all ports of call in Japan, Shanghai and Hong Kong; and from there to Manila. The experiences of such a trip are invaluable to any engineer, and the expense is ultimately borne by the Government. Otherwise it would cost him approximately $300 to $350. During the time consumed in the direct trip of thirty days, the appointee sees half the world and is on half salary. During services the Islands may be explored. When such service terminates, or vacation is granted, the return trip may be made by way of Europe. He thus completes the circle of the globe, arriving again in the United States with considerable experience and a good start in the world.

Climatic conditions are excellent, the temperature seldom falls below 65 degrees F or runs above 95 degrees F. A breeze is always to be encountered and the rainy season is of short duration. The sur-



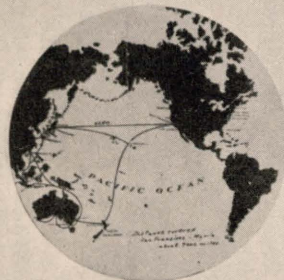
No 1 A corner of walled City [illegible] Manila P.I.

No 7 Typical Surveying Party [illegible] P.I.

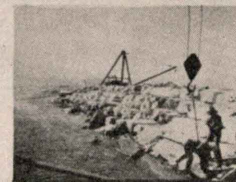


No 4 Placing of stone for breakwater Manila Bay Manila P.I.

No 2 Hydraulic dredge in distance and pipe line Manila, P.I.



"ROUTE"



No 5 [illegible] appearance breakwater Manila Bay Manila P.I.

No 3 Pipe line discharging into moat Walled City Manila P.I.

A floating palace S.S. Dakota
630 ft long 73 ft wide 28000 tons

No 6 Final touch on breakwater Manila Bay Manila P.I.

face of the Islands is in general far from swampy, mostly hilly and has an excellent natural drainage system in forms of creeks and rivers.

Life in the Islands is of a healthy out-of-doors nature. The houses are merely shelters from rain and sun, and in the rural districts are of an open type, elevated from five to ten feet above the ground, allowing a perfect circulation of air. They are equipped with shutters for protection against the rains and afternoon sun. This particular type of house is of native origin, and in material consists of bamboo and grass thatching. Another type is of Spanish architecture and is built entirely of heavy masonry and hard wood. The walls are three to four feet thick, being designed to withstand heavy earthquake shocks. The native type is designed to sway with the shock, eliminating the heavy strains, which would necessarily follow in a rigid structure. Food is abundant and of the best quality. Cold storage products, such as butter, meat, poultry and the like, are imported from the fertile lands of Australia, and are equal or better than the average dairy products put out in the U. S. In the rural districts water is always boiled as a safeguard against sickness, while Manila, the capital, enjoys the advantage of having one of the largest water distilleries in the world. Scarlet fever, diphtheria, and zymotic diseases common here, are seldom heard of. Yellow fever has never been known in the Islands. The species of mosquito necessary for the propagation of this disease does not exist there. The new General Hospital, with its able staff of physicians in connection with the Bureau of Science, offers care and medical attention to the sick unsurpassed by hospitals or institutions of its kind in the United States.

Social conditions are of the best. The engineer who has the pleasure of sharing Manila's society can feel that he has achieved something worth while, for the social circles of Manila are composed of men and women of intelligence, education and executive ability.

Engineering Projects of the Past.

One of the earliest problems presented to the engineer in the Islands was to perfect a system of sewerage and to obtain a pure water supply for Manila. In this task there were many difficulties to overcome. Manila is somewhat below sea-level, thereby preventing a sewerage system of the gravity type, and necessitating the installation of expensive pumping stations throughout the city to force the sewage far out into the bay. The water supply is obtained from the Mariquina river, patrolled by a constabulary guard to prevent contamination, and is pumped into a large reservoir at Montalbon, a few miles out of the city, and thence piped to the city. These were the first and greatest steps towards the sanitation of Manila. Next came the necessity of doing away with the mosquito germ-breeding Moat, which is a large canal around the walls of old Manila, or around what is termed the Walled-City, and is filled mostly with sewage. This was a big problem. Where could the material be obtained to fill in the millions of cubic yards of canal depths? At what cost? Such a problem, if carried out in the usual way, seemed next to impossible in consideration of the enormous expense. It was solved simultaneously with the question of a deeper harbor as follows: Bulkheads were built at various points in the moat (See photo No. 1) to hold within its walls any material other than water. At the bottom of these bulk-heads a sewer was provided to drain off all water running over or through the bulkhead. To provide the filling material for the moat, one which would displace the water, causing it to flow over the bulwarks into the drainage sewer was the next thing to consider. The bay of Manila had to be deepened. Where was the dredged material to be disposed of? Here was a case where the waste of one was the saving of the other. The Atlantic Gulf & Pacific Co. had the contract for this work, and their large hydraulic dredge was used to deepen the harbor. The dredgings of a half water and a half mud mixture were pumped through ball and socket jointed pipes floated on pontoons (See photo 2) to the point where it emptied into the moat (see photo 3).

The solid earth particles settled, the water drained off through the bulkhead, and thus the moat was filled ridding the city of its most dangerous source of mosquitos and at the same time giving it a deep harbor, permitting any ocean-going steamer to take protection within its breakwater. In a similar manner a large extension of land was made (See photo 2, to right) by building a sea-wall of stone and earth, pumping mud and sand into the enclosed portions.

Another very important improvement made by our engineers was the completion of the new breakwater. The work could only be carried on during certain seasons due to the heavy typhoons. Stone was quarried at Mariveles, Bataan Province, and floated on large scows to Manila Bay, and thence placed in position by large derricks (See photo 4). In the early part of the construction, the stone was dumped apparently promiscuously, but in reality, within certain controlling lines, and dumped so as to form a sloping pile, which, when reaching the surface, was built by layers, step-fashion (See photo 5). Finally a surface layer of stone was placed with a proper slope paved and the breakwater completed (See photo 6). This breakwater has saved many lives and thousands of dollars worth of property, and stands as a monument to the American Engineer.

Surveying in all its forms is practiced in the Islands. (See photo 7) and is enjoyable work in such a mild climate. The large Friar estates are being re-surveyed and divided; new railroads are being built of an up-to-date type; a complete trolley system has been installed in the city and is now being extended radially in all directions; small "Coney Islands" are being built; parking and city improvements in general, are in progress. One great piece of construction work consisted in the building of a fifty-mile road from the town of Dagupan to the summer capital, Baguio. This road is built on an upward grade twining among and along the sides of the mountains, and making leaps over large chasms. It was completed in 1907 and cost our government four millions of dollars. Other similar roads are in progress. Mining engineering is in its infancy and is making rapid strides. At present there are several large gold-mining concerns in operation and the controllers of these mines are making vast fortunes.

Bridges of all types and other structures are in process of erection. There are greater openings and more demands than ever for engineers along these and various other lines, and here lies the opportunity for our young college graduates.

The mistaken idea, that is borne in the minds of the Americans in general, seems to have shaped the Filipinos into man-eaters and guerrillas. Such ideas are greatly in error, for there could not be found a more peaceful and self-retiring individual than the Filipino In the mountains there are a few wild tribes, which, however, are seldom encountered. If the work of an engineer takes him through these regions ample protection is provided in the form of a military or constabulary guard.

It is evident from the facts, that an enormous amount of American capital is being poured into the Islands by capitalists; and by the vast amount of fortification work going on and the large navy yards which are built and being enlarged; and despite the rumors of immediate independence, "America has come to stay." Our engineers' future is therefore assured. It's up to you now. Are the doors of opportunity going to stand locked to you with the lock of ignorance or are you going to get the key of investigation and step inside?

PRATT TRUSS RAILROAD BRIDGE

S. B. EHENRICH, 1011, C. E.

1. The Pratt truss is one of the common types used for railway bridges, finding favor because it is simple of design and construction. It is pin-connected, and each member is subjected to but one kind of stress, with no reversals. The Pratt truss is used for spans of over one hundred feet. For extremely long spans the truss is designed with a variable depth, and is called the "Curved Chord Pratt Truss."

There are many methods for the determination of stresses in a Railway Pratt truss, four of which are given: (1) by algebraic resolution, (2) by the equivalent uniform loading for live load (live load treated conventionally), (3) by uniform loading plus a concentrated load for live load, and (4) by graphical resolution. The first method is the longest, but gives the most accurate results. In the second and third methods, the speed of the calculations is increased at the expense of accuracy. The last method is the most rapid, and the accuracy depends on the care and accuracy of the draftsman. These four methods will be compared by analyzing the same truss, using the following data:

Length of span, center to center of bearings.................225 ft.
Distance, center to center of trusses .... 17 ft.
Length of panels .... 25 ft.
Depth of truss (about 1/6 span) .... 37 ft.
Weight of track, floor, ballast, etc.........................600 lbs.

Live loading, two Cooper E-50 engines followed by a uniform train load of 5,000 lbs. per ft.

Alignment—tangent.

Grade—level.

Specifications—Am. Ry. Eng. and Main. of Way Assoc., 1910.

An odd number of panels is generally taken, as it simplifies the designing of the truss.

Algebraic Resolution.

2. Weight of bridge for E-50 loading $= 650 + 7$ x span, or 2,225 lbs. per ft. of bridge, or 500,625 lbs. for the whole bridge (see Malcolm's Graphic Statics, formula 3, page 215). One-third of this weight or 166,875 lbs., is considered as applied on the top chord, therefore the upper panel load is 1/8 of 166,875 or 20,900 lbs. The remaining two-thirds or 333,750 lbs. is applied on the lower chord, together with the weight of the track, floor, etc., or 600 x 225 or 135,000 lbs. The lower chord panel load $= 1/9 \ (333{,}750 + 135{,}000) = 52{,}000$ lbs.

The total dead load reaction = 1/2 x (8 x 20,900 + 9 x 52,100) = 318,050 lbs.

Effective reaction = 318,050 − 1/2 x 52,100 = 292,000 lbs.

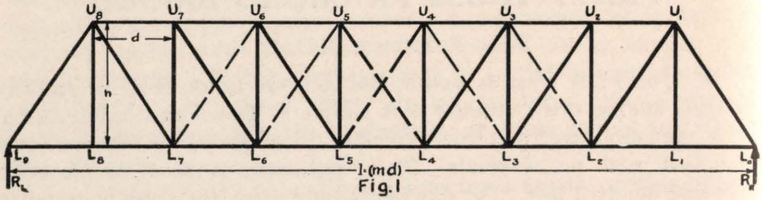


Fig. 1

3. **Notation.***

m = the number of panels in the bridge (= 9).

n = the number of the panel point in question, counting from the right to the left.

l = the length of span in feet, from center to center of bearings (= 225 ft.)

W ÷ 2 = dead load in pounds per foot of bridge for one rail.

Rl = Effective reaction in pounds at the left support.

Rr = Effective reaction in pounds at the right support.

d = length of panel, in feet. (= 25 ft.)

h = depth of truss, in feet (= 37 ft.)

V = vertical shear at panel point, in pounds.

S = Stress in member in question in lbs.

O = angle between diagonals and verticals, degrees.

Panel points are designated as shown in Fig. 1.

4. **Dead Load Chord Stresses.** To obtain the dead load chord stress cut that chord by a vertical plane MN and let = n be the panel point where the other two members cut cut by this plane intersect. Taking moments about n, to eliminate the moments of the two other cut members, S x h = Rl (m-n) d-w/2 ((m-(n + 1)) d × (m-n) d/2.

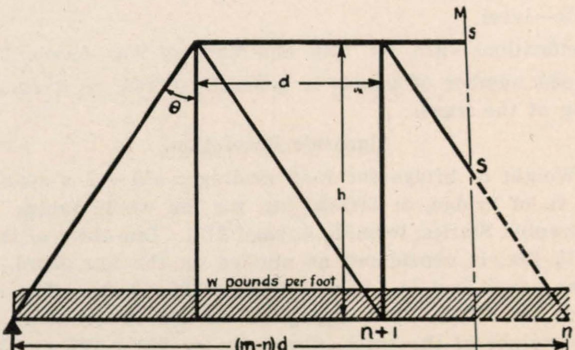


Fig. 2

Foot Note. *The notations and principal derivations are taken from "Roofs and Bridges", Merriman and Jacoby, by permission of the authors.

But $- Rl = (m-1/2)\ d\ w/2$.

$Rl = (m-1/2)\ d\ w/2$.

Therefore $S \times h = (m-l)\ (m-n)\ wd^2/4 - (m-n)\ (m-n-l)\ wd^2/4$.

$S = (m - n)n\ wd^2/4h$ ....(1)

Substituting for $wd^2/4h$, $S = 1193\ (m - n)\ n$.

It will then be seen that but half the truss need befigured, as the right half would give minimum values. The stress in the upper chord of any panel on the left half of the truss, is the same as that for the lower chord of the panel to the right. For if these two are cut by a plane MN, the vertical, or post, which is also cut, can take no horizontal stress. Consequently the lower and upper chords form a couple. The stress in the upper chords, $U_8\ U_1$ will be compressive, and in the lower chords $L_9\ L_0$ tensile. The bottom chord of the middle panels $U_6\ U_3$, will be equal, having the same point for the center of moments. The bottom chords of the first two panels, $L_9\ L_8$ and $L_8\ L_7$ act as a single member and their stresses are equal.

5. **Dead Load Web Stresses.** To obtain the dead load Web Stresses cut the member by a plane MN and the strss is then given by the formula derived below. If the member is a post, the plane should slope towards the center of the truss, to avoid cutting a diagonal; if a diagonal, the plane should be vertical. Here n is the panel point just to the right of the cutting plane. Web members take all the vertical shear, as the two chords cut by MN cannot take vertical shear.

$S = V \sec O$.

But—

$V = Rl\ (m - (n + l)\ dw/2$, and $Rl = (m - 1/2)\ dw/2$.

Eliminating V and Rl, $S = (2\ n + 1 - m)\ wd/4 \sec O$........(2)

This formula may be used for all web members but the hip vertical, $U_8\ L_8$. To the verticals a top panel load must be added, as it is not taken care of in the formula and it is also to the right of the cutting plane. The stress in the verticals is compressive, except in the hip vertical, which serves only to carry the lower panel load at L_8 to avoid flexure of $L_9\ L_7$, and is consequently in tension. All diagonals but the end posts are in tension. Reversals are taken care of by using counters wherever such reversals may occur.

For diagonals, $\sec O = (25^2 + 37^2)\ /37 = 1.207$, and $S = 21,311.1$ $(1\ (2n + 1 - m) -$ ....(2A)

For verticals, $\sec O = 1.000$ and $S = 17,655.3\ (2n + 1 - m) + 20,900$ .... (2B)

6. **Live Load Chord Stresses.** The live load is assumed as entering from the right. Let Ma be the moment of all live loads on the bridge about Rr, and Ml be the moment of all the live loads to the

left of panel point n about that point. Cut the member by a plane MN and let n be the panel point where the other two cut members intersect. Taking moments about n,

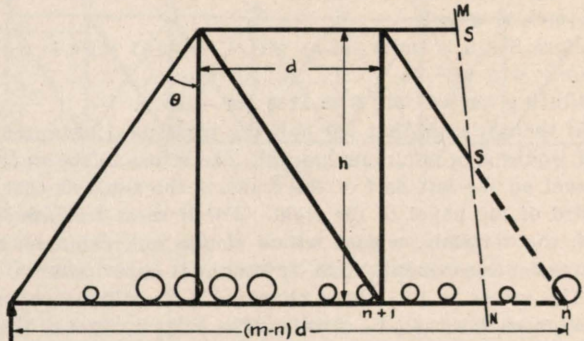


Fig. 3

$S \times h = Rl\ (m - n)\ d - Ml$.

Taking moments about Rr, $Rl \times md = Ma$, as $l = md$.

Substituting for Rl, $S = (Ma\ (m - n)/m - Ml)\ 1/h$ ....(3)

A wheel is placed over panel point n, and $Ma\ (m - n)/m - Ml$ is figured. Then the other wheels are tried, and the one found to give the greatest value is used. To do this for every wheel, would be a very long and tedious task. To assist in cutting down the time, and to give less chances for inaccuracy in figuring, a moment table has been devised from which moments may be easily determined.

7. **Key to Moment Table.**

Let P = sum of wheel loads on bridge, in pounds, considered as acting at the center of gravity.

g = the distance in feet, from the center of gravity of the wheel loads on the bridge, to the beginning of the uniform live load of w pounds per foot.

x = the number of feet of uniform live load on the bridge.

w = the weight per foot of the uniform live load (= 5000 lbs).

Then $Ma = P\ (g + x) + wx.x/2 = Pg + Px + wx^2/2$..........(4)

Referring to the moment table, (fig. 4), the loading is shown at the top, giving values for each wheel load, and for w. Each wheel is numbered, and the uniform load divided into ten-foot parts, and numbered. Below, opposite the word "dist." on the left, are the distances between the wheel loads. The next row gives three figures, for g, P, and Ma respectively as shown in the figure. If the first wheel is off the bridge, and wheel 2 is considered as o, the next row of figures is used, and so on, values being given for each wheel of the first engine at o. Thus for wheel 15, g = 88 ft.; P = 490,000 lbs; and Ma = 21,632,000 ft. lbs; x = o.

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	
Load	20	40	40	40	40	26	26	26	26	20	40	40	40	40	26	26	26	26	40	40	40	40	40	40	40	40	40	40	
dist.		8	5	5	5	9	5	6	5	8	8	5	5	5	9	5	6	5	5	10	10	10	10	10	10	10	10	10	10
d P M	0 20 0	8 60 160	13 100 460	18 140 960	23 180 1660	32 206 3280	37 232 4310	43 258 5702	48 284 6992	56 304 9264	64 344 11696	69 384 13416	74 424 15336	79 464 17456	88 490 21632	93 516 24082	99 542 27178	104 568 29888	109 568 32728	119 608 38608	129 648 44888	139 688 51568	149 728 58648	159 768 66728	169 808 74008	179 848 82288	189 888 90968	199 928 100048	209 968 109528
		0 40 0	5 80 200	10 120 600	15 160 1800	24 186 2640	29 212 3570	35 238 4872	40 264 6032	48 284 8144	56 324 10416	61 364 12036	66 400 13856	71 444 15876	80 470 19872	85 496 22222	91 522 25198	96 548 27808	101 548 30548	111 588 36228	121 628 42308	131 668 48788	141 708 55668	151 748 62948	161 788 70628	171 828 78708	181 868 87188	191 908 96068	201 948 105348
			0 40 0	5 80 200	10 120 600	19 146 1680	24 172 2410	30 198 3442	35 224 4432	43 244 6224	51 284 8176	56 324 9596	61 364 11216	66 404 13036	75 430 16672	80 456 18822	86 482 21558	91 508 23968	96 508 26508	106 548 31788	116 588 37468	126 628 43548	136 668 50028	146 708 56908	156 748 64188	166 788 71868	176 828 79948	186 868 88428	196 908 97308
				0 40 0	5 80 200	14 106 320	19 132 1450	25 158 2242	30 184 3032	38 204 4504	46 244 6136	51 284 7356	56 324 8776	61 364 10396	70 390 13672	75 416 15622	81 442 18118	86 468 20328	91 468 22668	101 508 27548	111 548 32828	121 588 38508	131 628 44588	141 668 51068	151 708 57948	161 748 65228	171 788 72908	181 828 80988	191 868 89468
					0 40 0	9 66 360	14 92 690	20 118 1242	25 144 1852	33 164 2984	41 204 4296	46 244 5316	51 284 6536	56 324 7956	65 350 10872	70 376 12622	76 402 14878	81 428 16888	86 428 19028	96 468 23508	106 508 28388	116 548 33668	126 588 39348	136 628 45428	146 668 51908	156 708 58788	166 748 66068	176 788 73748	186 828 81828
						0 26 0	5 52 130	11 78 442	16 104 832	24 124 1664	32 164 2656	37 204 3476	42 244 4496	47 284 5716	56 310 8272	61 336 9822	67 362 11838	72 388 13648	77 388 15588	87 428 19668	97 468 24148	107 508 29028	117 548 34308	127 588 39988	137 628 46068	147 668 52548	157 708 59528	167 748 66708	177 788 74388
							0 26 0	6 52 156	11 78 416	19 98 1040	27 138 1824	32 168 2514	37 218 3404	42 258 4434	51 284 6816	56 310 8236	62 336 10096	67 362 11776	72 362 13586	82 402 17406	92 442 21626	102 482 26246	112 522 31266	122 562 36686	132 602 42506	142 642 48726	152 682 55346	162 722 62366	172 762 69786
								0 26 0	5 52 130	13 72 546	21 112 1122	26 152 1682	31 192 2442	36 232 3402	45 258 5490	50 284 6780	56 310 8484	61 336 10034	66 336 11714	76 376 15274	86 416 19234	96 456 23594	106 496 28354	116 536 33514	126 576 39074	136 616 45034	146 656 51394	156 696 58154	166 736 65314
									0 26 0	8 46 208	16 86 576	21 126 1006	26 166 1636	31 206 2466	40 232 4320	45 258 5480	51 284 7926	56 310 8448	61 310 9998	71 350 13298	81 390 16998	91 430 21098	101 470 25598	111 510 30488	121 550 35798	131 590 41498	141 630 47598	151 670 54098	161 710 60998

MOMENT TABLE
2 - 142 TON ENGINES
COOPER'S "E" 40

Note:- Moment in thousand pound feet
Load in thousand pounds

Fig. 4

If the point for which these values are required, is under the uniform live load, and not at a point for which they are given, it can easily be shown that for P, the value for the first point to the left for which values can be given are used, using the corresponding value for g and the moment, Pg. Then x will be the distance between the two points. Take point 133 feet from wheel 1. Here $g = 129$ ft.; $P = 648{,}000$ ft. lbs.; $x = 133 - 129 = 4$ ft. and $Ma = Pg + Px + \frac{1}{2} wx^2$ (Equation 4) $= 44{,}888{,}000 + (648{,}000 \times 4) + (\frac{1}{2} \times 4{,}000 \times 4^2) = 47{,}512{,}000$ ft. lbs.

For Ml the third figure under the wheel in question is taken, as a wheel load is always placed at a point for which Ml is used.

8. To aid still further in the work, a criterion may be mathematically deduced, to determine which wheel to use. Let l′ be the distance between Rl and panel point n. Taking moments about Rr,

$Rl \times 1 = Ma = Pg + Px + wx^2/2$.

$Rl = (Pg + Px + wx^2/2)\ 1/l$.

Let $P' =$ the sum of the loads, in pounds, to the left of the panel point n.

$g' =$ the distance from the center of gravity of the loads to the left of panel point n to that point.

$l' =$ the distance, in feet, between Rl and n.

Taking moments about n.

$Ml = Rl' - P'g'$.

Substituting, $= (Pg + Pg + wx^2/2)\ l' \div 1 - P'g'$.

For Ml to be maximum, differentiate to zero.

$dMl/dx = l'/l\ (O + P + wx) - P' = o$ or $P' : P + wx :: l' : l$.

Or, for maximum moment at point n, the following proportion must hold true, the sum of the loads to the left of point n, is to the sum of the entire loads on the bridge, as the distance from the left support to point n, is to the span.

Take U_8U_7, and cut it by a plane MN, which also cuts L_8L_7 and U_8L_7. Then $n = 7$ as L_8L_7 and U_8L_7 intersect at L_7.

Then $l'/l = 2 \times 25/9 \times 25 = 0.222$. Wheel 6 gives $P'/P + wx = 0.214$, and wheel 7 gives it as 0.237. By using wheel 6, (Ma (m — n)/m — Ml) is 20,630, and wheel 7 gives it as 20,680, the greater one being used. The values given in the moment table are for E-40 loading, and are increased 25% for E-50 loading. These values are given also for both rails, so one-half of these values must be taken for each truss. Therefore 20,680 is multiplied by 1/37 according to formula (3), and by $5/4 \times 1/2$ for stress in one truss, giving $S = 349{,}570$ lbs.

9. **Live Load Web Stresses.** Let $Vn +_1$ be the vertical shear at point $(n + 1)$ then

$V = Rl - Vn + 1$.

But $Rl = Ma/md$ and $Vn + 1 = Ml/d$

Therefore $V = Ma/md - Ml/d = (Ma/m - Ml)\ l/d$

and $S = V \sec O = (Ma/m - Ml) \sec O/d$ ....(5)

Posts are figured for the left half of the truss only for maximum values. The diagonals are figured throughout, those on the right half giving stresses in the counters which are used to avoid stress reversals. The member is cut by a plane MN, and n is the point to the right of the cutting plane. Different wheels are placed at n to determine which one will give maximum shear at that point. A criterion may be deduced to determine this.

Taking moments about Rr

$Rl \times l = Ma = Pg + Px + wx^2/2$

Or $Rl = 1/l \ (Pg + Px + wx^2/2)$

Considering the panel $n + 1$ to n only, take moments about n,

$Vn +_1 \times d = P'g'$, or $Vn +_1 = P'g'/d$

Then $V = Rl - Vn + 1 = 1/l \ (Pg + Px + wx^2/2) - P'g'/d =$ $1/l \ (Pg + Px + wx^2/2 - P'g'm)$ as $l = md$.

For V to be the maximum, differentiate to zero.

$dv/dx = o + p + wx - mP' = o.$

$P/(P + wx) = 1/m = 0.111.$

Or, for maximum shear at point n, the sum of the loads to the left of n must equal one-ninth the sum of the entire load on the bridge. Take U_8L_9, and cut it by a plane MN, giving $n = 8$. Wheel 2 gives the nearest to the criterion, giving a value of 11,902,400 ft. lbs. for $(Ma/m - Ml)$. This is multiplied by sec 0/d according to formula (5) and by 5/4 and ½ for E-50 loading for one truss, giving $S = 359{,}060$ lbs.

MAXIMUM FLOOR BEAM REACTIONS, OR STRESS ON THE HIP VERTICAL.

10. The **Dead Load Stress,** as already stated, is one lower panel dead load, or 52,100 lbs.

11. **Live Load Stress.** The engine loads are placed in the first two panels, with a driver over panel point b, as shown in figure 5.

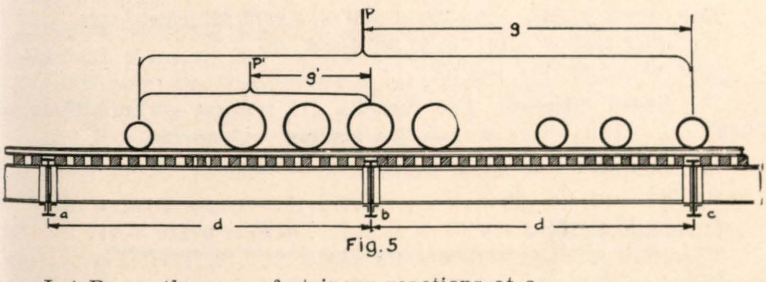


Fig. 5

Let Ra = the sum of stringer reactions at a.

Let Rb = the sum of the stringer reactions at b.

Taking moments about b, $Ra \times d = P'g'$ or $Ra = P'g'/d$.

Taking moments about c, $Ra \times 2d + Rb \times d - P \times g = o$.

Substituting $Rb = (Pg - 2P'g') \ 1/d$.

Let the moments of the loads to the left of $c = Mc = Pg$, and the moment of the loads to the left of $b = Mb = P'g'$, then $Rb = Mc - 2Mb)\ 1/d$ which is the required stress for both hip verticals. For each $S = (Mc - 2Mb)\ 1/2d$ ....(6)

$S = (Mc - 2Mb)\ 1/2d$ .... (6)

$= (5{,}702 - 2 \times 960)\ 1/37 \times 2 = 51{,}110$ lbs.

METHOD II.

Live Load Stresses Due to Equivalent Uniform Loading, Live Load Treated Conventionally.

12 Moments and shears are very easily determined for a uniform load, so in this method, a uniform load is substituted for the E-50 loading. To determine the uniform load w' which will be most accurate, the maximum bending moment is calculated for the E-50 loading at the quarter span, and this is equated with the maximum bending moment due to a uniform load at the same point. To determine the maximum moment for the E-50 loading, the criterion is $P'/P + wx = l'/l = 1/4$ for the quarter span. Taking moments of the loads to the left of this point.

Max. Mom. at quarter span $= Rl \times 1/4 - Ml$

But $Rl = Ma/l$.

Therefore Max. Mom. at the quarter point $= Ma/4 - Ml$.

Trying the different wheels, it is found that wheel 8 most nearly fulfiills the conditions, and that the moment is $\frac{1}{4} \times 140{,}256{,}400 - 7{,}127{,}500 = 27{,}936{,}600$ ft. lbs. for E-50 loading.

For w' uniform loading, $Rl = \frac{1}{2}w'l$, and $Ml = w'l/4 \times \frac{1}{8} = w'l^2/32$.

Taking moments at the quarter point,

Max. Mom. $= Rl \times 1/4 - Ml = w'l^2/8 - w'l^2/32 = 3w'l^2/32$.

Equating $3/32\ w'\ (225)^2 = 27{,}936{,}600$ ft. lbs.

And $w' = 5{,}900$ lbs. per ft.

13. **Chord Stresses.** Use formula (1) $S = (m - n)\ n\ w'd^2/4h = 24{,}909\ (m - n)\ n$ where $w'/2$ is the loading for one rail.

14 **Web Stresses.** $S = V \sec 0$.

$V = Rl = (1 + 2 + 3 + \ldots\ldots\ldots\ldots + n)/m\ w'/2d$.

Summation $(1 + 2 + 3 + \ldots\ldots\ldots + n) = n/2(n - 1)$

Therefore $S = n(n + 1)\ w'd/4m \sec 0$ ....(7)

For diagonals, $S = 4{,}945.4\ (n + 1)\ n$ ....(7A)

For verticals, $S = 4{,}097.2\ (n + 1)\ n$ ....(7B)

15. **Hip Vertical.** As for dead load, the hip vertical will carry one lower panel load or $w' \times d = 147{,}500$ lbs. for both, or 73,250 lbs. or each hip vertical.

METHOD III.

Uniform Loading Plus a Concentrated Load for Live Load.

16. In this method, a uniform load is used throughout the bridge, and one or more concentrated loads applied at various points. Different uniform loading and concentrated loadings will give different results, and the position of the latter is a factor. The most general practice is not to use more than one concentrated load in conjunction with the uniform load, and it is here assumed to act at the center of gravity of the wheel loads, or at a point 51.4 from wheel 1. This distance is found from the moment table, by using g = 109 ft. The distance of the center of gravity to 109 ft. will equal 32,728/568 = 57.6 ft., and therefore the distance from wheel 1 to the center of gravity is 1C9 − 57.6 = 51.4 feet. W″ or the uniform loading per foot is taken as 5,000 lbs.

The maximum moment for uniform loading is $3/32\ w'' \ l^2$ or 23,730,500 ft. lbs. Therefore the moment due to the concentrated load Q will have to be 27,936,600 − 23,730,500 or 4,206,100 ft. lbs. (26,-936,600 being the maximum moment at the quarter point for wheel loads, as determined in paragraph 12). When the bridge is fully loaded (to give the maximum moment), Q will be 51.4 ft. from Rl and 225/4 − 51.4 or 4.85 ft. from the quarter point. Taking moments about Rr.

$Rl \times 225 = Q\ (225 - 51.4)$, and $Rl = 173.6//225\ Q$.

Taking moments about the quarter point,

$Rl \times 225/4 - Q \times 4.85 = o$.

Substituting, $Q = 109{,}100$ lbs.

17. **Chord Stresses.** Chord Stresses due to uniform loads = $(m - n)\ n\ w''d^2/4h$ (Equation1). For Q take moments about Rr, Q being at point n.

$Rl \times md = Q \times nd$.

$Rl = Qn/m$.

Taking moments about n, $S \times h = Rl\ (m - n)\ d = (m - n)\ ndQ/m$ for both sides, or $S = (m - n)\ ndQ/hm$ for each side.

Combining the two stresses, $S = (m - n(\ n\ (wd''d^2/4h + dQ/2hm) = 25{,}210n\ (m - n)$ .... (8)

18. **Web Stresses.** The stresses due to W″ loading = V sec 0. For V to be the maximum, the load must come a distance x past point u.

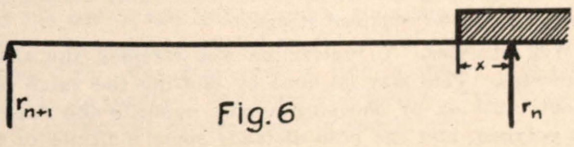


Fig. 6

Considering the panel only,

$V = Rl - Vn + 1$.

Taking moments about Rr, $Rl \times l = w\ (nd + x)^2/2$ and $Rl = w\ (nd - x^2/2l.)$

Taking moments about n considering the panel only

$Vn + 1 \times d = wx^2/$ and $Vn + 1 = wx^2/2d$.

Substituting. $V = w\ (nd - x)^2/2l - wx^2/2d$ .... (9)

For V to be maximum, differentiate to zero.

$dV/dx = w/2md\ (o + 2nd + 2x) - w2x/2d = o$, or $x = nd/(m - l)$

Substituting for x in (9)

$V = w/2md\ (n^2d^2 + 2n^2d^2/(m\text{-}l) + n^2d^2/(m\text{-}l)^2 - wn^2d^2m/2d\ (m\text{-}l)^2$

Simplifying, $V = n^2wd/2\ (m\text{-}l)$ for both sides or $V = n^2wd/4\ (m\text{-}l$

Taking moments about Rr, $V \times md = Q \times nd$ and for one side, $V = Qn/2m$.

Uniting total $V = n^2wd/4\ (m\text{-}l) + Qn/m$

and $S = (n^2wd/4\ (m\text{-}l) + n\ Q/2m)\ \mathrm{Sec}\theta$ .... (10)

For diagonals, $S = 4,714.8n^2 + 7,315.6n$ .... (10A)

For verticals, $S = 3,906.25n^2 + 6,061.0n$ .... (10B)

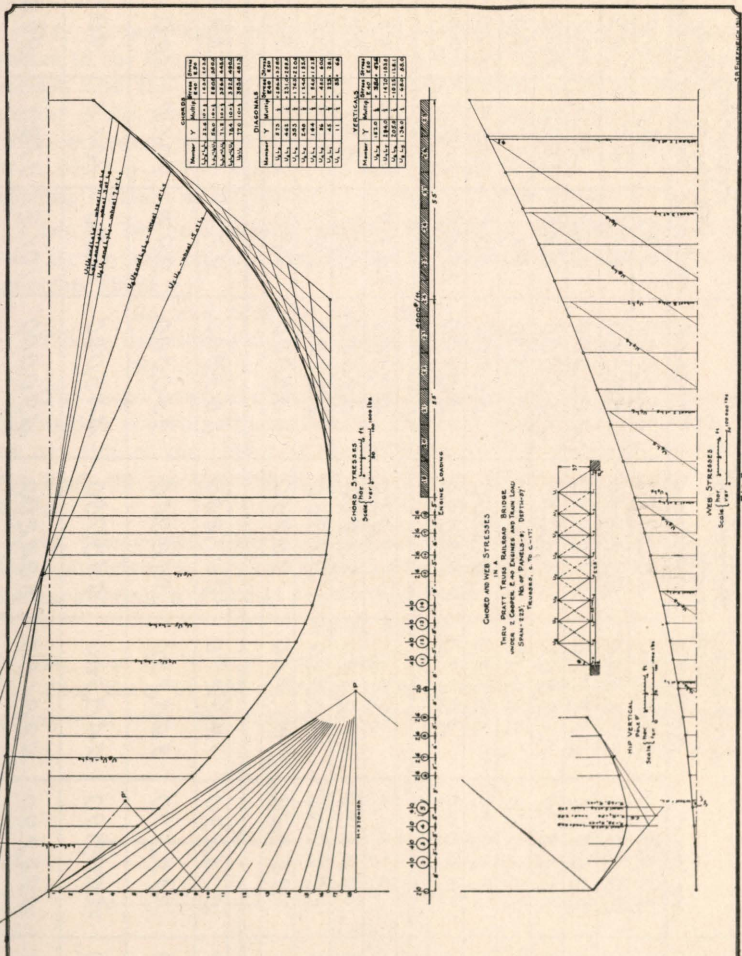
19 **Hip Vertical.** The stress in the hip vertical will equal one panel load plus the concentrated load or $S = \frac{1}{2}\ (w''\ d + Q) = 117,050$ lbs., a value very far in excess of its true value.

METHOD IV. GRAPHICAL RESOLUTION.

20. This method will be described briefly with reference to Figure 7. Stresses were computed for E 40 loading and increased 25% for E-50 loading. The space diagram is first drawn, and then the force diagram, using a proper scale to keep the drawing within limits. A pole is chosen opposite the last wheel load, to assist in drafting, and the pole distance made some multiple of the depth of the truss, H, here the multiple chosen was 10.

21. **Chord Stresses.** The rays of the force polygon are drawn, and the funicular polygon constructed for the wheel loads. For the uniform load, the curve will be a parabola, and may be drawn in by the method indicated in the figure. To determine the stress in a chord member, the bridge is moved so that the panel point, (which is the same as in the other methods) is at a wheel load. Perpendiculars are erected at the end of the truss, to cut the curve and those points connected. The ordinate between this line and the curve at the point in question is scaled. The same is done for other wheels, and the one giving the maximum ordinate, is used. This value multiplied by the pole distance gives the required stress.

22. **Web Stresses.** To determine web stresses, the shear curve must be plotted. This may be done by plotting the value of P from the moment table or by choosing a pole opposite the first point of the force polygon, and the pole distance some multiple of the span. From this curve, if drawn to the proper scale, the reactions may be read for any position of the live load. To obtain the stress in a web



member, the bridge is moved, so that a wheel is over the panel point (which is determined as for the other method), and a perpendicular erected at the end of the bridge, which gives the reaction of Ma/Md. Then Ml/d is figured and scaled down, giving stresses for verticals, sec O being 1.00. For diagonals, a line is drawn from the point parallel to the diagonals in the truss and scaled. The curve is drawn to such a scale that the stresses may be read at once.

23. The following table gives the stresses for all members as figured by formulae here deduced.

Member		Dead Load Stresses in Pounds	Live Load Stresses in Pounds Method 1	Method 2	Method 3	Method 4	Accuracy Method 1 = 100.00 Method 2	Method 3	Method 4
Diagonals	U_8L_8	170,480	379,060	356,070	360,270	358,000	99.2	100.3	99.7
	U_8L_7	127,860	290,620	276,940	282,230	288,800	95.3	97.1	99.3
	U_7L_6	85,240	220,250	207,710	212,630	220,600	94.3	96.5	100.1
	U_6L_5	42,620	154,470	138,360	154,450	155,000	89.6	100.0	100.3
	U_5L_4	0	103,000	98,910	104,000	102,500	96.0	101.6	99.5
	U_4L_3	0	59,970	59,340	61,380	60,000	98.8	102.4	100.1
	U_3L_2	0	28,270	29,670	33,490	28,100	104.9	118.5	99.4
Verticals	U_8L_8	52,100	51,110	73,250	117,050	47,500	143.9	230.9	93.0
	U_7L_7	91,520	199,780	172,080	173,990	183,800	86.1	87.1	92.0
	U_6L_6	56,210	127,980	122,920	127,960	128,100	96.1	99.9	100.1
	U_5L_5	20,900	85,330	81,940	92,750	85,000	96.0	108.7	99.6
Chords	L_9L_7	95,540	205,230	199,270	201,680	203,800	97.1	98.3	99.3
	$\left.\begin{matrix}L_7L_6\\U_8U_6\end{matrix}\right\}$	167,020	349,570	348,730	352,940	350,000	99.8	101.0	100.1
	$\left.\begin{matrix}L_6L_5\\U_7U_6\end{matrix}\right\}$	214,740	446,890	448,360	453,780	445,000	100.3	101.5	99.6
	$\left.\begin{matrix}L_5L_4\\U_6U_4\end{matrix}\right\}$	238,600	492,290	498,180	504,200	490,000	101.2	118.1	99.5

Stringers.

24. Stringers are placed 6′ — 6″ center to center, and are connected to the floor beams. Stringers carry the track, ties, guard rails, ballast, and the weight of stringers themselves, as dead load. The weight of the track, floor, ballast, etc., is 600 lbs. per foot, the weight of the stringers is assumed as 175 lbs. per foot, making the dead load $W = 600/_2 + 175 = 475$ lbs. per foot for one stringer. The dead load bending moment at the center $= \frac{1}{8}\ wd^2 = 37{,}110$ ft. lbs.

Let Mc be the maximum bending moment of the center due to live load on the stringer (see Fig. 5), and Ra be the reaction at a. Taking moments about b.

$Ra \times d = Ma$ or $Ra = Ma/d$

Taking moments about the center of the stringer.

$Mc = Ra \times d/2 - Ml = Ma/2 - Ml.$

The usual position of wheels for Mc is to have the four drivers of the engine covering the stringer. The second driver is to be as far on one side of the center of the stringer, as the center of gravity of the loads on the stringer is on the other. This gives $Mc = 375{,}000$ ft. lbs.

25. Dead load shear $= \frac{1}{2}\ wd = 5{,}940$ lbs. For live shear, place the first driver at a, and taking moments about b, $Ra = 70{,}650$ lbs. for each stringer, which is the vertical shear.

Floor Beams

26. Let Pd be dead load, and Pl be the live load transmitted to the floor beam by the stringers. $Pd = 26{,}050$ lbs., or half a lower panel dead load (see paragraph 10) and $Pl = 51{,}110$ lbs. (paragraph

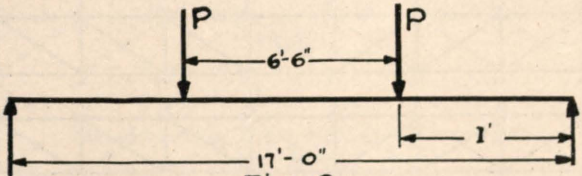


Fig. 8

11).

The dead load bending moment at the center will equal $Pd \times l^1 = 26{,}050 \times \frac{1}{2}\ (17 - 6.5) = 136{,}760$ ft. lbs. The live load bending moment at the center $= Pl \times l^1 = 293{,}880$ ft. lbs.

27. The dead load shear at the end $= Pd = 26{,}050$ lbs. The live load shear at the end $= Pl = 51{,}110$ lbs.

Wind Load Stresses.

Statement of Wind Loads.

28. The wind load on each of the top and bottom lateral systems is taken as 200 lbs. per linear foot of truss, to be treated as dead load,

half being applied on each truss. The live wind load is taken as 600 lbs. per linear foot of bridge, of which 150 lbs. per foot is assumed as acting on the bottom chord, and 450 libs. per foot is assumed as acting six feet above the base of rail, or eleven feet above the center of the

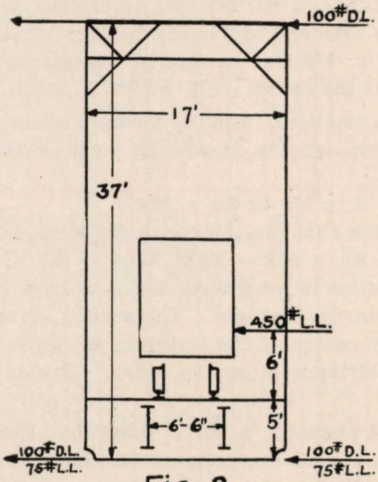


Fig. 9

bottom chord, half of the former applied on each truss. Fig. 6 shows these loads in magnitude, direction and points of applications, for wind from the right.

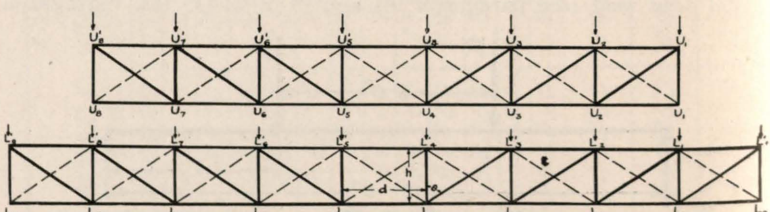


Fig. 10

29. **Bottom Lateral System.** (a). Dead wind panel load = 200 × 25 = 5,000 lbs. The bottom lateral system is treated as a Pratt truss, and stresses due to the dead wind load caluculated by the same formulae (1) and (2), for chord and web members. The depth h, and the angle Q will change, h = 17 ft. and sec Q = 2.212,

Stress in chords = (m — n) n $wd^2/2h$ = 3,676.5 (m — n) n..(11)

Stress in diagonals = (2n + 1 — m) wd/2 sec Q = 5,530 (2n + 1 — m) ....(12A)

Stress in vertical = (2n + 1 — m) wd/2 = 2,500 (2n + 2 — m) .... (12B)

30. (b) Live wind panel load = 600 × 25 = 15,000 lbs.

Stress in chords = 300% of dead wind load stress given by (11) .... (13)

Stress in diagonal s = n (n + 1) wd sec Q/2m = 1,843.3 n (n + 1) (as equation (7) .... (14B)

Stress in vertical s = n (n + 1) wd/2m + 525d = 833.3n (n +1) 13,125 .... (14B)

31. The following table gives the dead and live wind stress in the bottom lateral system, as calculated from formulae (11) to (14B).

Diagonals.

Member	L_9L_8	L_8L_7	L_7L_6	L_6L_5	L_5L_4	L_4L_3	L_3L_2	L_2L_1
Dead load stress (lbs)	44120	33180	22120	11060	0	0	0	0
Live load stress (lbs)	132720	103220	77420	55300	36870	22120	11060	3690

Verticals.

Member	L_9L_9	L_8L_8	L_7L_7	L_6L_6	L_5L_5
Dead load stress (lbs.)	22,500	17,500	12,500	7,500	2,500
Live load stress (lbs.)	67,500	54,160	42,500	32,500	24,170

Chords.

Member	L_9L_8	L_8L_7-L_9L_8	L_7L_6-L_8L_7	L_6L_5-L_7L_6	L_5L_4-L_6L_4
Dead load stress (lbs.)	0	29,410	51,470	66,180	73,530
Live load stress (lbs.)	0	28,230	154,410	198,540	220,590

32. **Top Lateral System.** Stresses are calculated by the same formulae as for the bottom lateral system, differing only in the value of n, as there are less panels. The results are given by the following table.

Web Members.

Member	U_8U_7	U_7U_6	U_6U_5	U_5U_4	U_8U_8	U_7U_7	U_6U_6	U_5U_5
Stress (lbs.)	33180	22120	11060	0	175000	12500	7500	2500

Chords.

Member	U_8U_7	U_7U_6-U_8U_7	U_6U_5-U_7U_6	U_5U_4-U_6U_4
Stress (lbs.)	0	22,060	36,770	44,120

Stress in Main Truss Due to Overturning.

33. The wind load of H lbs. per foot on top chord. Let V = Reaction at one end post due to overturning, in pounds.

H = Horizontal force in pounds per foot of top lateral system.

b = Distance from center to center of trusses, in feet. Taking moments about a, see fig. 11.

$$V \times b = \frac{H\ (m-l)\ d \times h}{2}$$

$$V = Hd\ (m - l)\ h\ 1/2 = 21{,}764.7 \text{ lbs.}$$

The overturning moment due to wind on truss only, affects stress in the top chords U_8U_1, bottom chords L_9L_0, and in the end posts U_8L_9 and U_1L_0 only.

Chord Stresses = V tan Q = 14,710 lbs.

End Post Stresses = V sec Q = 26,170 lbs.

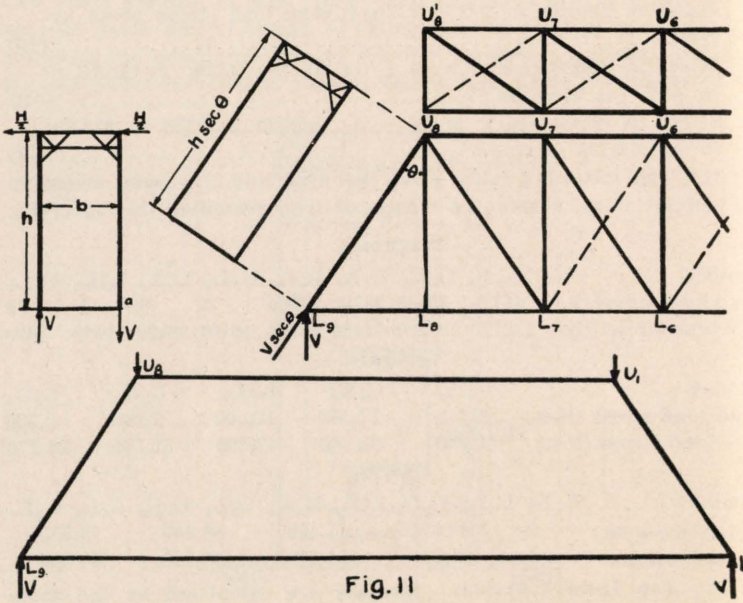


Fig. 11

34. (b) Wind of H lbs. per foot on train. Taking moments about a, $V \times b = H \times s$ where s = distance betwen the point of application of the wind, and the bottom lateral system = 11 ft.

$V = H\ s/b = 264.7$ lbs., and stresses due to V are treated as uniform live load conventionally.

Stress in chords $= (m - n)\ n\ Vd^2/2h = 2{,}235.4\ (m - n)\ n$ see equation (1) .... (15)

Stress in verticals $= n\ (n + 1)\ Vd \sec Q/2m = 443.87\ n\ (n + 1)$ (see equation 7A) .... (15A)

Stress in verticals $= (n + 1)\ Vd/2m = 367.7n\ (n + 1)$. (See equation 7B) .... (15B)

The stresses calculated by these formulas are as follows:

Diagonals.

Members	U_8L_9	U_8L_7	U_7L_6	U_6L_5	U_5L_4	U_4L_3	U_3L_2	U_2L_1
Stress (lbs.)	31950	24900	18640	13310	8880	5320	2660	890

Verticals.

Member	U_8L_8	U_7L_7	U_6L_6	U_5L_5
Stress (lbs.)	20,590	15,440	11,430	7,350

Chords.

Member	L_9L_7	L_7L_6-U_8U_7	L_6L_5-U_7U_6	L_5L_4-U_6U_4
Stress (lbs.)	17,880	31,300	40,240	44,710

35. The following table is a summary of stresses in the main truss, with maximum and minimum stresses:

Table 5.

Stresses in lbs. due to	End Post	Diagonals						Verticals				Chords			
	U_8L_9	U_8L_7	U_7L_6	U_6L_5	U_5L_4	U_4L_3	U_3L_2	U_8L_8	U_7L_7	U_6L_6	U_5L_5	L_9L_7	L_7L_6 U_8U_7	L_6L_5 U_7U_6	L_5L_4 U_6U_4
Dead Load	-170480	+127860	+ 85240	+ 42620	0	0	0	+ 52100	- 91620	- 56210	- 20900	± 95540	+167020	±214740	±238600
Live Load (I)	-359060	+290620	+220260	+154470	+103000	+ 59970	+ 28270	+ 51110	-199780	-127980	- 86330	+205230	±349570	±446890	±492290
Wind Overturning on train from S	- 3	+ 24900	+ 18640	+ 13310	+ 8880	+ 5320	+ 2660	- 20950	- 15440	- 11430	- 7350	+ 17880	± 31300	± 40240	± 44710
Wind Overturning on train from N	+ 31950	- 24900	- 18640	- 13310	- 8880	- 5320	- 2660	+ 20950	+ 15440	+ 11430	+ 7350	- 17880	∓ 31300	∓ 40240	∓ 44710
Wind Overturning on truss from S	- 26170											+ 14710	± 14710	± 14710	± 14710
Wind Overturning on truss from N	+ 26170											- 14710	∓ 14710	∓ 14710	∓ 14710
Maximum	-587660	+443380	+324130	+210400	+111880	+ 65290	+ 30930	+124160	-306740	-195620	-113580	+333360	±562600	±716380	±790310
Minimum	-112360	+102960	+ 66600	+ 29310	- 8880	- 5320	- 2680	- 31150	- 76080	- 44780	- 13550	+ 62950	±121010	±159790	±179180

THE ENGINEERING ANNUAL OF THE CIVIL ENGINEERING SOCIETY, VALPARAISO UNIVERSITY.

When the Civil Engineering Society decided unanimously to publish a technical annual,, it was necessary to amend the constitution of the society in order to provide for the continued publication of this annual. The amendment which was adopted,, and upon which the management of the annual is based is as follows:

Amendment to the Constitution of the Civil Engineering Society.

Resolved, That the Civil Engineering Society of Valparaiso University publish annually a bulletin to be known as "The Engineering Annual" of the Civil Engineering Society of Valparaiso University, or such other name as may hereafter be determined by a two-thirds vote of the total active membership of the Society.

Resolved, That the purpose of this annual is to publish articles on Civil Engineering subjects by professors, alumni and under-graduates of this University.

Resolved, That the management shall be in the hands of three editors: Editor-in-chief, exchange editor, and financial or business manager, to be elected by the society from members of the Senior and Junior classes at the first business meeting of the second term. The editor-in-chief and exchange editor shall each select one assistant and the financial manager shall select two assistants. The editor-in-chief shall be a member of the Senior class, while the exchange and financial editor may or may not be. But there must be at least one Junior member on the board, who shall serve for two years, becoming editor-in-chief upon his entrance to the Senior class. Should there at any time be two Junior members of the board, the editors and their assistants shall decide which one is to become editor-in-chief.

Duties of the Editors.

The editor-in-chief shall have complete charge of the publication. He shall attend to the business of soliciting articles suitable for the purpose of the annual. He shall criticise all manuscripts submitted, such criticisms to be referred to the Advisory Board, whose personnel and duties are hereinafter described. The editor-in-chief shall, at the end of each term and at the last regular meeting of that term, make a complete report to the society of the progress of the work.

The exchange editor shall ascertain with what societies the annual may be profitably exchanged, and the number of copies of the annual necessary to be printed. The societies with which publications are to be exchanged shall be determined by vote of the board of editors and their assistants. The exchange editor shall report to the editor-in-chief, who shall communicate to the financial manager, the contents of this report.

The financial manager shall act as treasurer of all moneys paid to the account of the annual. He shall conduct a business campaign for the purpose of securing advertisements. He shall supervise the arrangement of advertising matter. He shall collect and pay out all money. He shall make a written report to the editor-in-chief and thru him to the advisory board, one week after publication has been issued. Be it further

Resolved, That there shall be an Advisory Board to consist of the Dean of the School of Civil Engineering and the Professor of Civil Engineering, whose duties shall be to advise the board of editors on the management of the magazine. They shall act on the criticisms on manuscripts referred to them by the editor, and shall make their recommendations to him. They shall also pass on the acceptability of all matter, both articles and advertising.

Resolved, That the society shall be assessed for the purpose of defraying the expenses of the annual, other than that covered by money received from subscription and advertising. All surplus money in the hands of the financial manager after all the expenses of the annual have been paid, shall be turned in to the treasury of the so. ciety.

Civil Engineering Society.

Organization: The Civil Engineering Society of Valparaiso University came into existence in the fall of 1909, Prof. R. C. Yeoman being the moving spirit in its foundation. At first the membership was limited to upper-classmen, though at present this policy has become outgrown, and now membership is offered to any member of the University upon payment of the regular membership fees.

Object: It is the purpose of the society to stimulate and foster among its members an intelligent and appreciative interest in their chosen profession. To accomplish this, engineers of well known professional standing are invited to speak before the society from time to time, their subject being chosen generally from their own experience. Addresses are also made by the under-graduates who have had any outside experience.

Meetings: Meetings are held on the first and third Fridays of the month. After the speaker of the evening has surrendered the floor, there is a general discussion of the subject during which the members are offered an excellent opportunity of obtaining at first hand a clear conception of the conditions which he is preparing himself to meet.

From the opening of college in the fall of 1910 the following program has been carried out:

List of Subjects and Speakers.

Oct. 7, 1910—"Cost Analysis," by S. R. King.

Nov. 4, 1910—"The Manufacture of Steel and Iron", by R. C. Yeoman, Professor of Civil Engineering.

Dec. 16, 1910—"Highway Construction", by S. B. Ehrenrich, C. E., 1911; Engineer with the New York State Highway Commission.

Jan. 6, 1911—"Theater Construction", by C. W. Hinshaw; "Opportunities for Young Engineers", Geo. Coughlin.

Jan. 20, 1911—"Engineering Aspects of Illustration", by N. S Amstutz, Research Engineer; Investigator and Originator of System of Photography by Wire.

Feb. 3, 1911—"The Comstock Lode," by Herbert Muller, C. E.

Feb. 17, 1911—"Steel and Concrete Inspection", by H. B. Johnson, Engineer with the Robt. W. Hunt Company.

March 30, 1911—"Draughting Room Organization and Its Relation to the Shops", by H. C. Gregg of the Gary Steel Mills.

April 14, 1911—"Coal", by Lee F. Bennett, Professor of Geology and Mineralogy, Valparaiso University.

Among the probable speakers who will address the society in the near future may be mentioned Prof. O. P. Kinsey, Vice-President of the University, and Prof. M. E. Bogarte, Dean of the School of Civil Engineering.

Officers—First Term.

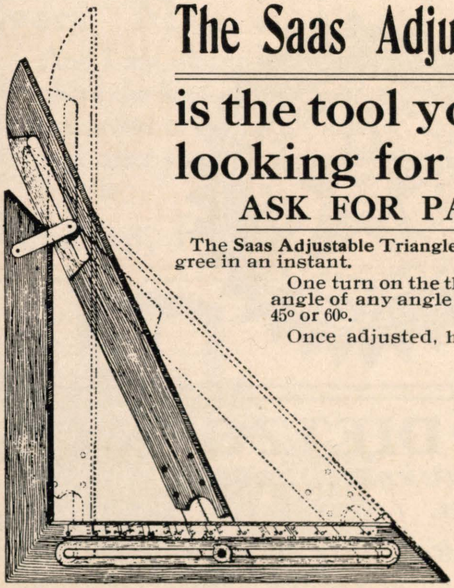
President .... Herbert Muller
Vice-President .... Emil Oeffinger
Secretary .... R. D. Jones
Treasurer .... E. A. Hurme

Second Term.

President .... Arthur D. Buzby
Vice-President .... S. B. Ehrenrich
Secretary .... E. C. Earle
Treasurer .... E. A. Hurme

Third Term.

President .... S. B. Ehrenrich
Vice-President .... E. C. Earle
Secretary .... Arthur D. Buzby
Treasurer .... E. A. Hurme



The Saas Adjustable Triangle

is the tool you have been looking for all the time.

ASK FOR PARTICULARS

The **Saas Adjustable Triangle** can be adjusted to fit any degree in an instant.

One turn on the thumbscrew and you have a triangle of any angle you **need**, instead of the usual 45° or 60°.

Once adjusted, holds its position firmly, until another adjustment is required.

It is flush on either side, and lies flat on the paper, as well as on the T square.

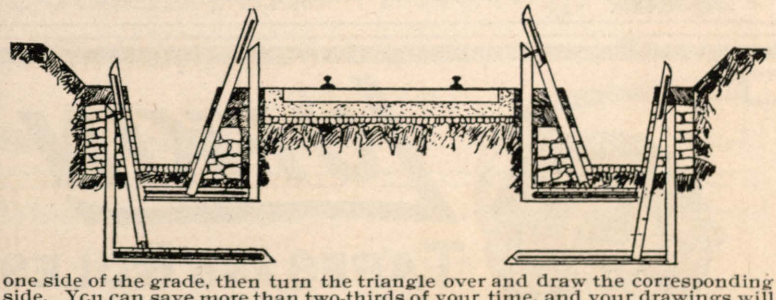
There are no better and quicker means to transplant angles from one board to another. There is not a branch of the profession, where you don't need the help of this triangle.

PATENTED

On the basis of the **Saas Adjustable Triangle** is marked:

1. **Every half degree from** 0° **to** 45°.
2. **The following** 11 **slopes:** 1:1:, 1:1½, 1:2, 1:2½, 1:3, 1:4, 1:5, 1:6, 1:8, 1:12, and 3:4.
3. **The slopes of the letters: A. K. M. N. V. W. X. Y.**

In making plans for roadbeds or highways, do not use cross-sectional paper any more. There is no reason, why you should measure the slopes with the cross sections thousand and thousand times, on both sides of the embankment, when you can set your triangle to the desired slope in an instant, draw



one side of the grade, then turn the triangle over and draw the corresponding side. Ycu can save more than two-thirds of your time, and your drawings will be neater and more accurate.

A simple instrument, admirably designed and well built. A necessity to every drafting room.

Adjustable Triangle Company

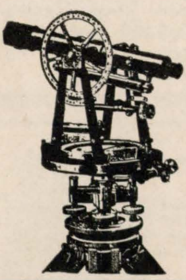
7 E. 42nd St. **New York City**

ELMER STARR
470 COLLEGE AVE.
CLOCKS POSTERS SPOONS PINS FOBS POST CARDS STATIONERY PENNANTS SPORTING GOODS PILLOWS
TREAT U RIGHT

EUGENE DIETZGEN CO.

MANUFACTURERS

166 West Monroe St. - - - - - Chicago

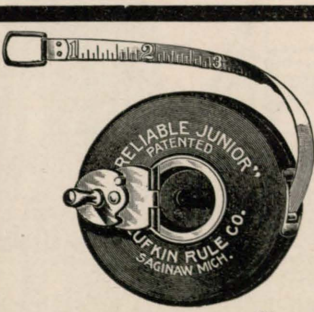


Drawing Instruments
Surveying Instruments
Field and Office Supplies

ILLUSTRATED CATALOG FREE

NEW YORK SAN FRANCISCO NEW ORLEANS
PITTSBURG TORONTO PHILADELPHIA

Factories: Chicago and Nurenburg



LUFKIN

TAPES AND RULES

ARE THE BEST IN THE WORLD.

Made by **THE LUFKIN RULE CO.**

Saginaw, Mich., U. S. A.
New York. London.

FOR SALE EVERYWHERE.
SEND FOR CATALOG